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DEVOTED TO CIVIL ENGINEERING AND CONTRACTING

F. E. SCHMITT, *Editor*

Volume 103 NEW YORK, NOVEMBER 14, 1929 Number 20

Irresponsible Statements

OCCASIONALLY men in responsible executive positions express their views on current matters in such a way as to make it appear that the views expressed are those of the organization which they represent. A case in point is a recent attack upon the Corps of Engineers by a vice-president of American Engineering Council. The fact that the speaker is an official of the Engineering Council was prominently displayed in the press reports. Had his official position not been made known to the reporters, it is safe to assume that the views expressed would never have gotten into the papers. The publication of such an unofficial pronouncement is particularly unfortunate in the case of the Engineering Council because its sole function is that of spokesman for the engineers of the country. Since the public does not discriminate between official and unofficial statements, it is urgently important that officers of the council confine their public statements to subjects upon which the council has taken official action. If Engineering Council is to come up to the high ideal set for it, it may not be used to air personal views.

Chicago Again Defeats Bonds

STEADILY increasing objections to higher taxes, together with a deep-rooted distrust of the present city and county administrations, have resulted in renewed defeat of a number of bond-issue projects and a tax increase proposal by the voters of Chicago and Cook County. Out of nearly $47,000,000 of bond-issue proposals, only $3,000,000 were approved at the election on Nov. 5. The defeat of three city bond issues totaling nearly $19,000,000 was unfortunate, since the money is needed to pay judgments for damages to property owners on street-widening work already done. Civic associations had recommended a favorable vote on the items for damages. At least one organization had advised voting for all the proposals except the $20,000,000 West Park District elevated boulevard. With so varied a program, sweeping generalizations as to the cause for the adverse votes would be risky, yet distrust of the city and county governments is unmistakably evident. There seems to have been more confidence in some, at least, of the various park commissions, for the one proposal that carried was for a modest expenditure to enlarge and improve Lincoln Park. Continued defeats of bond issues in Chicago and Cook County give rise to hopes for a general housecleaning at the next election.

Behavior of Embankments

UP TO NOW we have had no such data on the settlement of high embankments as has been compiled for the two New England earth dams in the record published on another page. The amount of settlement is less surprising than its long continuance; and the difference in respect to this period between the two dams also invites attention. But the record as a whole is satisfying in so far as it gives fuller knowledge of high masses of earth and shows that in relatively few years they settle to a state of compactness which no doubt represents a lasting condition. It may be gathered that normal earth dams are relatively least stable when new, and improve by consolidation as they grow older, so that save for special agencies of deterioration or attack they acquire greater safety with the passing years. These considerations do not apply to spillway and conduit structures, but engineering skill is competent to deal safely with these elements when once all question as to the permanent security of the embankment body is out of the way. Knowledge of the behavior of earth masses has been slowly accumulating in recent years, to excellent effect; the present data make an important addition to the stock.

A Missing Link

SUCH glaring faults in structural steelwork as are pointed out in this issue cast reproach on the structural profession and on the fabricating industry which serves it. The plans for some of the poorly built structures described may originally have shown a design of unquestioned adequacy, but somewhere in the history of the project departures were made—and excused on the pretext of economy and in the knowledge that skimped construction would get by. Somewhere there is a missing link in our building control methods that makes these departures from safe and adequate design possible. The fault may lie with the city building department in not providing suitable coverage by its inspectors. It may be possible because the building ordinance requires nothing more than approval of plans. Again, perhaps the fault is more deeply rooted, for many men are posing as engineers and architects who have only a mediocre technical knowledge and no professional integrity. What is the solution? Mr. Eden suggests licensing fabricating plants. If professional licensing laws are only partly effective, attempts to extend licensing to the commercial field of steel fabrication appear hopeless. The solution, it appears, must be sought in a revitalized professional conscience and an aroused public opinion—the former to tolerate no connection with work of doubtful safety or excellence, the latter to demand and enforce professional supervision of engineering construction.

Complex Framework

THAT high standards of excellence in framework construction are essential to public safety—though the proposition is really self-evident—finds strong emphasis in such complex examples of modern building design as the two-theater building described in this issue. Heavily loaded long spans were once rare in building constructions, but they have become commonplace. And the commonplace in steel-building work is in danger of

being treated by routine procedure, with danger of neglect, instead of enlisting highly skilled engineering attention which these complex structures deserve. Though the elements of steel framing are simple, yet they may occur in highly intricate combinations where stresses are difficult to appraise and where variations in workmanship seriously affect the interaction of the members and joints; and again, such structures, being costly, often tempt designers or fabricators to harmless-looking economies which may produce dangerous weakness. Architects and owners do not always recognize the grave responsibilities inherent in safe structural design, if one may judge by the frequent haggling over engineering fees in the building field. A definite engineering responsibility for structural safety, enforced by public building officials and extended over construction as well as design, would go far toward assuring that high degree of safety which is essential to modern building.

A Double Negative

VOTERS at last week's election in San Francisco registered decided approval of an incinerator as a means of disposing of the city's garbage, but rolled up a substantial majority against both the operating propositions that were proposed, of which one provided for municipal operation and the other for private operation. The city fathers wanted an expression from the people as a guide to the preferred method of solving the garbage problem, but the form in which the matter was put before the voters presented no clear-cut issue nor did the voters have any detailed information as a basis for judging the relative merits of the proposition submitted. In the matter of incinerator operation the city fathers appear to be rather worse off than before the vote, since the voters have said, in effect, "Yes, we want incineration; but we do not want the incinerator operated by the city nor do we want it operated by private interests." Since these two latter expressions negative one another, the supervisors have the operating question back in their own hands, where all of these problems really belong. Fortunately, five new supervisors came into office as the result of this same election, and since the vote demonstrated the fallacy of looking to the people for a solution of garbage-disposal problems it is possible that there will now be direct action by the city officials. Unless it does, the court that has been withholding a public-nuisance injunction against the operation of the present incinerator may lose patience.

The Competitor Pays

ALMOST unusual line of reasoning is employed by the Inland Waterways Corporation in a plea to the Interstate Commerce Commission for a more favorable division of rates between rail and barge lines. The corporation asks that the railways be required to accept a smaller share of joint rates than they now receive, and justifies its action by claiming that the railways really will not suffer from this reduction in income because in aiding the barge lines they are strengthening the country's transportation system and so aiding in upbuilding the general prosperity of the country. Under present conditions the railroads receive 10 to 20 per cent less for their proportion of a joint-haul rate where the freight is turned over to a barge line for part of the trip than they do when they turn it over to another railroad to complete the movement. Thus not only does one railway lose the shipment entirely but another railway takes less for its proportion of the return from the freight movement in order to help keep the barge line in business. But the Inland Waterways Corporation is not satisfied with this arrangement. It wants the minimum differential raised to 20 per cent. Such a proposal is beyond reason. The government now subsidizes the barge lines. It should not call upon the railways with which they compete to add further to the subsidy.

Shall Superhighways Be of Single or Dual Pavement Strips?

SO MANY multiple-lane highways are being built in metropolitan areas that engineers as well as users, roadside residents and business interests are becoming interested in the merits of different types of these wide pavements. Most of these highways are created by widening existing two-lane pavements, through the addition of one or more lanes, and the result necessarily is a single wide roadway. In Milwaukee County, Wisconsin, however, boulevard construction—traffic lanes separated by parking—has been used to some extent, and the system has found many ardent advocates.

Safety and traffic-capacity arguments speak in favor of the Milwaukee or split-slab system, with neutral ground separating the opposing lines of traffic. It eliminates head-on collisions and the glare of lights for night drivers. In a solid multiple-lane road many drivers ride along the center line out of willful inconsiderateness, and many others out of timidity—fear of dropping off the edge of the slab if they stick to the extreme right. The timid driver is perhaps the worse evil of the two, for he is likely to be a slow driver and in consequence invites pernicious passing on the right or cutting into the forbidden opposing lane on the left. Both practices are dangerous, but without them traffic would be measurably slowed down. Rules for slow and fast lanes can probably be understood and enforced more easily on the separated system; the inner lane can be used for speed without the disadvantage of being directly adjacent to the opposing high-speed lane. The system also provides safe parking space in the center, on level ground, so that it becomes unnecessary to drive into the ditch when changing a tire or else to indulge in the dangerous practice of keeping the inner wheels on the pavement.

On the Blue Mound road, Milwaukee County, there is a long stretch of the boulevard construction terminated at either end by a stretch of solid multiple-lane pavement. Statistics of traffic on this road indicate that the two separated pavements carry more traffic, and faster by 10 to 20 miles per hour, than does the solid pavement at either end.

One major disadvantage of the separated-slab construction is that it costs more and requires a wider right of way. Also, until the dead strip in the center is well seeded and perhaps landscaped, it will not have a particularly pleasing appearance. Again, the crossing problem for residents is more serious. Drainage requires special treatment, particularly if no conduit system exists. The separate system is not so elastic in providing for peak loads of one-way traffic; but to balance this there is the fact that on few single-pavement systems is the prevailing traffic allowed to utilize more than one-half of the roadway. Much more attention needs to be paid to adapting lane use to traffic—that is, to provide workable means for devoting three lanes of a four-lane pavement to the heavy morning or night rush. Not only definite

marking but probably also effective policing is necessary for this purpose.

The Milwaukee split system will be watched with considerable interest by both Cook County, Illinois, and Wayne County, Michigan, where large mileages of 40-ft. pavement already exist and where much more is to be built. Lack of right-of-way, undoubtedly, will be a controlling factor in choice of type in many cases, especially through territory already built up. On the other hand, the constantly increasing urge for building more inherent safety into highways regardless of cost should make highway designers investigate every possibility with open mind and examine conditions on existing types by accurate survey.

The boulevard system is unquestionably more pleasant to drive over. The sense of safety is greatly enhanced at all times, but especially at night. Whether comparative statistics of accidents will confirm this feeling remains to be learned. Definite figures to express relative safety will be important to the road designer in selecting type, for safety and capacity are perhaps the two dominant factors wherever feasibility and cost do not limit the choice.

Lake Levels and Shore Protection

WITH Lake Michigan at an exceptional but not unprecedented high level, a series of severe storms on the lake during the past few weeks has caused remarkably extensive damage to public and private property along the entire lake frontage of Chicago. The losses, though scattered, reach so large a total as to approach the dimensions of a disaster. Park beaches, drives, grounds, buildings and shore-protection works have been washed out, eroded, flooded or damaged. Quantities of sand and mud have been deposited on park lands and drives by the heavy waves, and residences and apartment buildings have been flooded or have had their foundations threatened by the shore erosion.

Rough estimates of the damage to public property have amounted to more than $1,000,000 for repair work alone. But the larger question is how further erosion and destruction may be prevented, and how the needed funds for such new work are to be provided. Even in the repair and reconstruction work, it is evident that the engineers of the park commissions and other public authorities will have to revise their plans and not simply rebuild on the original lines. Jetties, bulkheads, breakwaters (high and submerged), and even a row of sand-fill off-shore islands, have been suggested tentatively as protective works, but there is as yet no concerted action or definite plan. The financing of any extensive new works will be a difficult problem. As to private property, owners of improved or valuable buildings and sites must in many cases foresee the necessity for more substantial and effective protection than that which has been provided hitherto. And this, of course, involves large cost.

Reviewing lake conditions and shore protection, it is seen that the present high level of the lake is only one contributing factor in the damage and destruction. Higher levels have been reached in past years. An important factor is the great development of municipal property and facilities as well as of private property along the lakefront and on reclaimed land. Where erosion was formerly of little importance, it now threatens damage or total loss to land and buildings. Further, there has apparently been a general failure to recognize the certainty of periodical high-water elevations, with consequent failure to provide adequate protection against high water itself and the storms which must be expected to accompany it. This is evidenced by inadequate height of new works above water level and by protective works of insufficient height and strength to resist the forces and undermining action of heavy storm waves. As far as private property is concerned, there has been also a tendency to the use of cheap and simple protection, a neglect of repair and maintenance and a lack of united action which might result in adequate protective works at reasonable cost.

Forecasts have been made that the present high lake level will continue through 1930, with possible increase to an even higher stage. Accordingly there is likelihood of much additional damage before Lake Michigan begins its next recession to lower levels. Under such conditions it is desirable that prompt action be taken to guard against further damage and to carry out repairs in such a way as to insure that the rebuilt works or structures will be safe against future storm attacks. How this is done will be watched with interest not only in Chicago but by all engineers interested in shore protection. Furthermore, study of this problem may present opportunities for developing new ideas and new designs in shore-protection works of greater effectiveness and economy.

The Public Deserves Protection

ANOTHER avoidable tragedy due to building hoisting operations was recorded in our news columns last week. An innocent passer-by was killed and several others were injured when a skip of facing stone being hoisted on a Chicago building job tipped at the 34th-floor level, spilling its entire two-ton load into the street below. That such an accident was possible is direct evidence of the usual disregard of public safety in building construction by contractors and by municipal authorities. As we have repeatedly stated in these pages, hoisting material over the heads of the public constitutes an inexcusable hazard and should not be tolerated. Derrick failures alone are common enough to render all hoisting operations dangerous, and with the countless other sources of hoisting accidents prevailing, such as in the Chicago case, the situation becomes all the more serious.

One sure precaution to the public is to stop all traffic from passing under the load during the few moments it is in the air. Although in every city the police have the power to do this as a public safety measure, the contractor should take the initiative in demanding it. If this procedure is impractical because of local conditions, other means should be resorted to in protecting the public. Hoisting stone and other small material in elevator cages rather than in open skips, and hoisting as much steel as possible at night, are within the range of possibility.

Hoisting alone is not the only danger the public is subjected to in building operations. Material trucks rush in and out of buildings without regard for pedestrian traffic. Temporary footwalks are inadequate, poorly lighted and cluttered up with building material. Many times, open holes are poorly protected. Material is piled unnecessarily on the walks, forcing pedestrians to take to the street. In constructing a modern building at the pace demanded of him the contractor must necessarily discommode the public. But the public has certain paramount rights, chief among them the right to safety. Owner, contractor and municipal authorities must regard this right at all times. At present all these parties are negligent in this respect.

Two Theaters in Chicago Office Building

Civic Opera and Civic Playhouse Will Be Provided For in New 45-Story Structure on Wacker Drive—Trusses Spanning Main Auditorium Carry the Building Columns for the Upper 37 Stories of the Tower

By E. A. Magurn
Chicago, Ill.

OF THE many new buildings which are changing Chicago's skyline, one of the most notable is an office building which houses both the civic opera and the civic theater, a condition which presented special problems in design, construction and equipment, while the building is intended to provide sufficient rental revenue to support the opera organization. Thus this 45-story structure, known as the 20 Wacker Drive Building, is partly superimposed upon two theaters with their auditoriums, stages and lobbies, in addition to the lobbies serving the office floors. The main and smaller theaters will seat about 3,600 and 800 persons respectively, while the opera stage, some thirteen stories in height, is said to be one of the largest in the world. A view of the river side of the building is shown by Fig. 1, and a cross-section through the main theater and wings by Fig. 2, while the plan, Fig. 3, shows the locations of the several steel trusses.

FIG. 1—45-STORY OFFICE BUILDING WITH THEATERS, CHICAGO
View of riverfront. Opera house in lower portion. Theater in north wing (at left). Wings, 23 stories. Main portion, 45 stories, 560 ft. high.

Problems in the structural design included high column loads, unsymmetrical arrangement of columns at different elevations, and trusses and girders to span large open spaces. The difficulties of design and construction were increased by the necessity of having the building ready for the opera season in the fall of 1929, although the preparation of plans and clearing of site did not begin until early in 1928. This building is 391 ft. long with a width of 189½ ft. at the south end and 150½ ft. at the north end. Its main portion has 23 stories, while the upper or tower portion, 71¾x239½ ft., extends to the 45th story and has its roof 560 ft. above the street. The total load on the foundation, including live loads, is about 202,000 tons.

For the civic opera, the auditorium is 158x122 ft., with a height of 80 ft. 9 in. to the ceiling at the seventh floor level. The proscenium arch is 50 ft. wide and 36 ft. high, with a stage 120x72 ft. having a total height of 157 ft. to its roof at the thirteenth floor level, or a clear height of 144 ft. to the suspended gridiron floor from which the scenery is handled. In the north wing is the civic theater, with an auditorium 80x57 ft. and 47 ft. in height. Its stage, 70x32 ft., has a height of 81½ ft. to the roof, or 71½ ft. to the gridiron.

Foundations and River Wall—All foundations carrying the main columns were extended to bedrock, about 100 ft. below water level, or 115 ft. below the street. However, the foundations for some columns under the auditoriums and the stages were carried down only to hardpan at 60 ft. below datum. Concrete piers of former buildings on the site were utilized in some cases, new piers being carried down alongside of the old ones and the column loads distributed between the two by plate girders.

A concrete mix of 1:1:2 was used in the piers going to rock and 1:2:4 in those carried to hardpan, as these proportions were found to be the most economical. For instance, the largest pier going to rock, 11 ft. in diameter, would require to be 13¼ ft. in diameter if the leaner mix were used, while the extra cost of digging would be considerably greater than the extra cost of the richer mix used in the 11-ft. pier. For the piers carried down only to hardpan, the situation is reversed, since the size could not be reduced with the richer mix, as a 4-ft. diameter is the minimum for digging. As large quantities of sand and water were found on top of the rock, light steel sheetpiling in lengths of 15 to 30 ft. was driven by hand, for lining the bottom portions of the wells, instead of the 4-ft. lengths of wood lagging ordinarily used.

Along the riverfront, before foundation work was started, a cofferdam was built, closing against the foundations of the Madison St. and Washington St. bridges, so that the wells along this front were constructed without any special trouble from river water. Difficulty was encountered, however, from remnants of old foundations and former docks. The old dock wall had been pushed about 2 ft. out of line, and old timbers, sheathing and rubbish behind it had to be cleared away before the concrete wall could be built.

To brace the river wall against impact from vessels, cantilever struts of reinforced concrete were introduced (see Fig. 4) to support the dock and distribute the lateral pressure to the interior piers. The concrete dock, 5 ft. wide by 8 ft. deep, was poured in blocks about 100 ft. long, the joints being sealed with a No. 24

gage soft copper stop placed 4 in. below the surface and the joint above this filled with an asphaltic composition. The river wall proper is 3 ft. thick. A compound was used in the concrete for these walls to increase workability and thus produce a dense concrete. The dock wall was carried down to 20 ft. below datum, or about 4 ft. below the river bottom at the dock line, thus leaving a seal of earth to prevent undermining by the backwash from steamers, which might result in hydrostatic pressure under the basement floor.

Structural Design—Steel frame construction is used throughout and the upper portion of the building was designed as a typical office building, having hollow-tile floors and concrete fireproofing on all steelwork. With the special layout of the theater facilities below and the practically uniform layout of the office floors above, problems of offsetting columns and transferring loads by means of heavy members were inevitable. In numerous cases the column loads have been carried over from one location to another, and in order to transfer these loads, it was necessary to introduce 121 heavy girders and seventeen trusses (see Fig. 3).

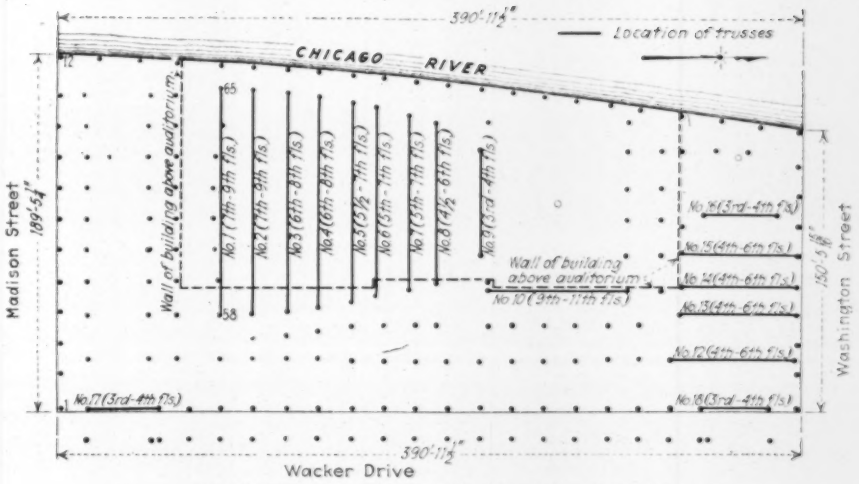


FIG. 3—PLAN SHOWING LOCATION OF TRUSSES

Trusses 1 and 2, seventh to ninth floor; 3 and 4, sixth to eighth floor; 5, 6 and 7, fifth to seventh floor; 8, 4½ to sixth floor; 9, third to fourth floor; 10, ninth to eleventh floor; 12 to 15, fourth to sixth floor; 16, 17 and 18, third to fourth floor.

Below the 44th floor the columns are carried down, in general, to the 23d floor. There they change location and are carried down to the ceiling of the auditorium, where they are supported by trusses whose supporting columns extend directly to the foundations. In the south wing it was necessary to introduce another set of girders at the seventh floor in order to provide for wide column spacing in the foyer. In the north wing the columns are carried from the roof down to the fourth floor, which forms the ceiling of the civic theater, where they are supported by trusses whose columns are carried down to the foundation piers.

On account of the requirement for large open spaces without columns in the auditoriums, stages and lobbies, it was sometimes necessary to concentrate several column loads on one lower column. This resulted in unusually high loadings, the maximum being 7,500,000 lb. on one foundation. Circular steel slabs were used as column bases. The largest is 8 ft. 2 in. in diameter and 13½ in. thick, weighing about 14 tons.

Trusses spanning the main auditorium had to carry

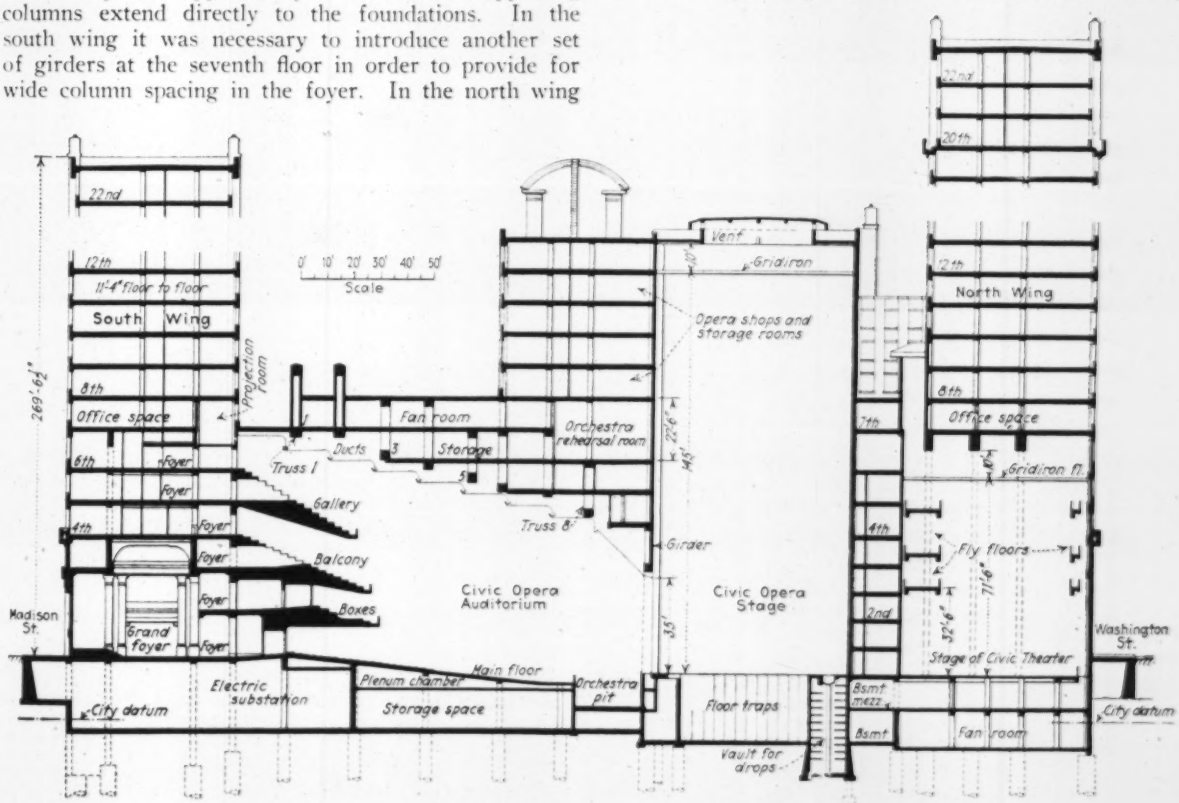


FIG. 2—SECTION THROUGH OPERA HOUSE AND OFFICE WINGS

the building columns for the upper 37 stories of the tower. The load on one end of these trusses is much heavier than on the other, thus requiring different types of details at the ends. In taking care of the high reactions involved, it was found that riveted end connections would introduce high secondary stresses with a tendency toward bending the columns. It was decided, therefore, to use pin and rocker shoe bearings. Truss No. 10 (Figs. 3, 5 and 6) supports five columns and has at one end a compression member coming in to the pin with a load of 5,700,000 lb. and another of 1,200,000 lb., together with the tension member carrying 2,300,000 lb., making a total reaction of 6,300,000 lb. The total load on this truss is 11,000,000 lb. A 20-in. pin was used, resting in a steel casting. This truss, with a span of only 73 ft., weighs 235 tons and one of its members weighs 40 tons. For the eight trusses (Nos. 1 to 8) of 90 to 120 ft. span over the main auditorium, the loads range from 3,200,000 lb. on No. 1 to 7,400,000 lb. for No. 6. On account of the size

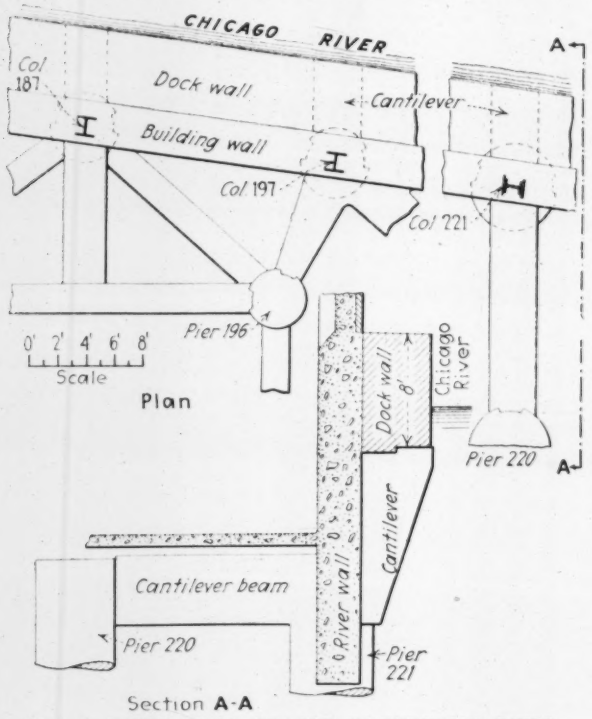


FIG. 4—RIVER WALL BRACED TO INTERIOR PIERS

and weight of the members these trusses were erected in the field.

Wind Bracing—This was figured for 30 lb. per square inch on the tower and 20 lb. on the main building, which allows for a theoretical wind velocity of 85 miles per hour. Top and bottom shelf connections were used (Fig. 7), cut from wide-flange I-beams. Where these were not sufficient, gusset plate brackets were used. It was necessary to provide these special flange wind-brace connections on both the interior and the exterior columns across the narrow width of the tower from east to west, while only the exterior columns of this upper portion are braced in the longer north and south direction.

The offset of columns and the change of location of resisting members at the 23d, 13th, and 7th floors made horizontal wind bracing necessary, the diagonals being flats 8x½ in., while the floorbeams and girders

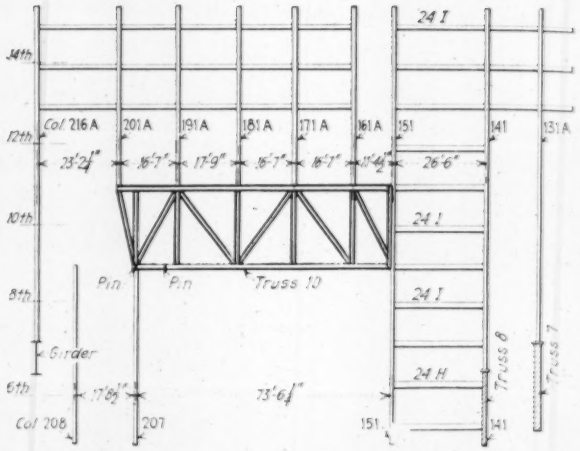


FIG. 5—TRUSS CARRIES FIVE COLUMNS

form the chords. Special bracing at each floor was also necessary on either side of the main auditorium, due to the open spaces or courts required along the sides by the city building ordinance. These courts are intended as a safety provision in case of fire and in them are the fire escapes. The columns in the walls of the auditorium are braced by struts, inclosed in concrete, at each floor level. Vertical diagonal wind bracing was used only in the rear wall of the stage of the civic opera and extends from the basement to the thirteenth floor level.

Balcony Framing—The opera auditorium has a mezzanine and two other balconies with truss and cantilever supports. In order to preserve the sight lines within the limited headroom, much engineering skill and ingenuity were required. Thus the cantilevers supporting the mezzanine balcony floor had to be interlocked with the fulcrum girders, allowing the tension member to pass through. As the cantilevers could not be supported on top of the fulcrum girder, the compression at this girder was taken care of by a tie-plate passing under the bottom of the girder (see Fig. 7). In the framing for the mezzanine balcony, diagonal girders are used to support the fulcrum girder, which is thus made only

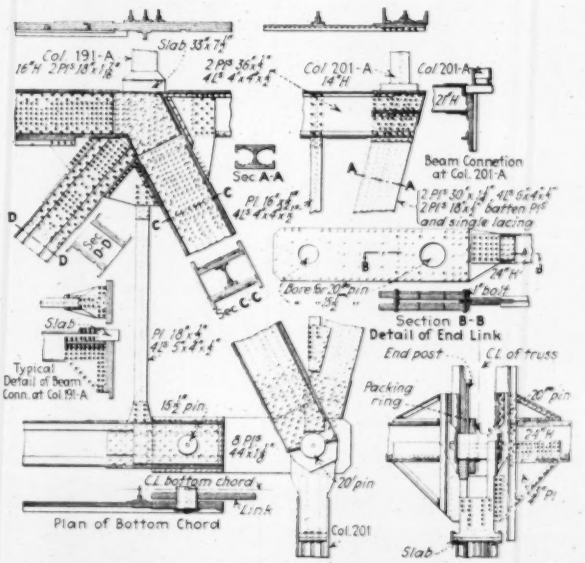


FIG. 6—TRUSS WITH RIVETED AND PIN CONNECTION

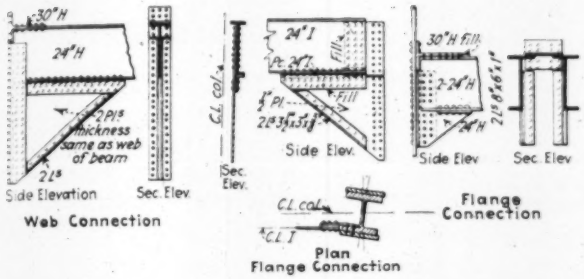


FIG. 7—TYPICAL WIND BRACING

84 ft. 9 in. long, reducing the deflection under load (see Fig. 9).

Heating and Ventilation—Direct radiation for heating the building will be supplied by a two-pipe vacuum system supplemented by combined heating and ventilation from fans. This equipment is designed to maintain a temperature of 70 deg. in all heated portions of the building when the outside temperature is 10 deg. below zero. All portions of the opera house and theater are to be ventilated, there being six changes of air per hour in the opera space outside the auditorium, and ten changes in the boiler room and other portions of the mechanical plant. The entrances, vestibules and lobbies are to be supplied with fresh air at the rate of 47,000 cu.ft per minute.

Fresh air for the opera auditorium, drawn from a light court at the eighth floor level and passed through filters, will be admitted at the ceiling and exhausted at the floor. This ventilation is to be accomplished without interference with the accoustics. To prevent vibration or noise from fans and motors they will be mounted over sheets of compressed cork. The total volume of air supplied to the main auditorium will be at the rate of 30 cu.ft. per person per minute, making a total air change of 8,220,000 cu.ft. of air per hour.

Elevator Equipment—There are to be 35 elevators, most of them of the variable voltage, high-speed gearless traction type. Ten elevators will extend to the fifteenth floor; eight express elevators will serve the fifteenth to 22d floors, and eight tower elevators serving the 22d to the 42d floors will run express from the first to the 22d. Most of these elevators will have a capacity of 2,500 lb. at a speed of 600 to 700 ft. per minute. There will be three balcony elevators from the first to the sixth floor; three opera elevators from the basement to the seventh floor, and a special 5,000-lb. elevator, 20x8 ft., for scenery, to run from the basement to the ninth floor level. A freight elevator will serve the entire height of the building and a private elevator will run between the 42d and 45th floors.

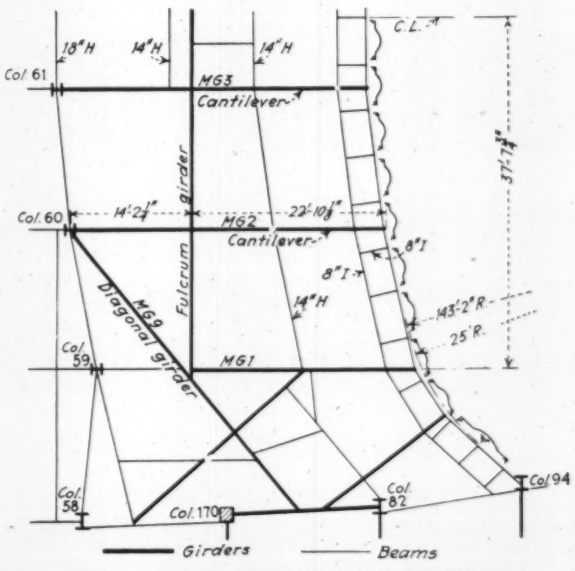


FIG. 8—CANTILEVER BALCONY FRAMING

Stage Features—A steam curtain behind the second row of footlights will provide for vapor or cloud effects. Three rows of sliding floor sections roll away under the stage floor to right and left, giving one opening of 2½x30 ft. and two openings 4x50 ft. A section of the stage floor 50x40 ft. is equipped with sectional traps 4x25 ft. which sink 12 ft. below the stage floor and rise 12 ft. above it and can also be tilted to an angle of 30 deg. These traps are controlled from a station at one side of the stage. This area is also equipped with sliding floor sections, so that if any trap is dropped below the stage level, the space can be filled with sliding floors, leaving such openings as may be desired.

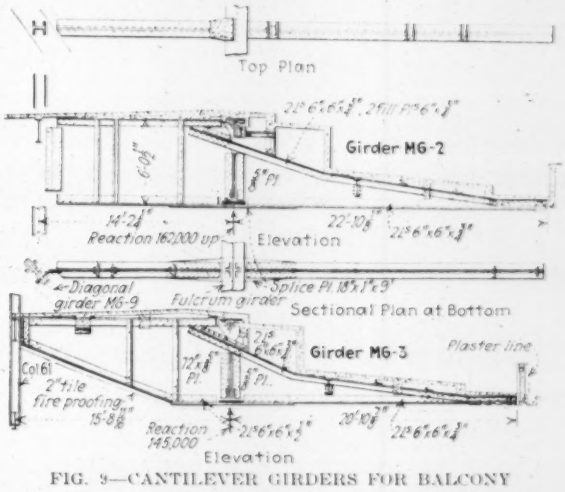


FIG. 9—CANTILEVER GIRDERS FOR BALCONY

Along the rear wall of the stage is a section of floor 5x75 ft. which is locked in position during performances but can be utilized as an elevator to handle scenery stored in a vault below the stage. Another elevator is provided for handling stage properties. All these elevators and traps are operated hydraulically from a pumping station equipped with three multi-stage centrifugal pumps, air compressors, pressure and exhaust tanks and piping connections. This system also provides pressure for water scenes and for operating the steel curtains. Behind the act curtain are two bridges 85 ft. long, equipped with spotlights. These bridges travel in guides and can be set in any location for proper lighting.

Miscellaneous Features—With an electric substation located in the basement, provision for the hoisting and placing of heavy electrical equipment is made by a hoisting eye in the portico roof and capable of supporting a load of 50,000 lb., removable slabs being provided in the sidewalk beneath. At 28 ft. below water level there is a connection with the Chicago Tunnel Company's system, which will serve the boiler room for coal delivery and removal of ashes. This connection has a gate to protect the building from water, fire or smoke in the tunnel. Coal will be delivered to a 500-ton storage bin

by conveyors fed by chutes from wagons in the street or from tunnel cars.

Fire-protection equipment includes pumps, tanks and a sprinkler system. One pump will have a capacity of 1,000 gal. per minute operating against a 200-lb. discharge pressure, to serve the main building, while the pump to serve the tower will have a capacity of 500 gal. per minute against a 300-lb. discharge pressure. A large automatic fire vent in the roof over the stage is arranged to open automatically in case of fire and thus keep flames and gases from spreading into the auditorium.

Designers and Builders—Both the architectural and the engineering designs were handled by Graham, Anderson, Probst & White, with Magnus Gundersen as chief structural engineer. The general contractor was the John Griffiths & Son Company, while steel erection was let to the McClintic-Marshall Company.

Operation of Sewage-Works at Princeton, Ill.

Small Activated-Sludge Plant With Mechanical Aerators Proves Satisfactory—Test Results Are Given

A YEAR'S operating results of the 500,000 gal. daily activated-sludge sewage-works at Princeton, Ill., were outlined at the last annual meeting of the Illinois Society of Engineers by F. C. Roe, of Shields, Jordan & Roe, Chicago, engineers for the plant. Activation is produced by the Simplex system, in which mechanical aerators bring the sewage into contact with the air at the surface of the tank. The works consist of a coarse screening chamber, pump house, preliminary settling tank, five aerating and two final settling tanks, a sludge-digestion tank and a sludge-drying bed. The detention period in the preliminary settling tank is one hour, and in the aerating and final settling tanks it is eight and two hours respectively. Traveling scrapers collect the sludge in the preliminary tank for pumping to the digestion tank. Similar scrapers collect the activated sludge, part of which flows to the first aerating unit and part to the digestion tank.

Operation began Jan. 20, 1928, and in spite of zero weather during the first few weeks the sludge built up rapidly and a stable effluent was produced. In the early spring, flows were recorded as high as 1,500,000 gal. daily after rains. The average flow up to May 1 was about 700,000 gal. daily. Since the plant was designed for 500,000 gal. daily, an overflow weir was placed between the preliminary settling tank and the aeration tank to allow all excess sewage to be discharged directly to the creek, but this excess sewage had the benefit of effective preliminary tank treatment. Since May 1 the entire flow has been treated except during periods of heavy rains, and has averaged close to the plant capacity. The extreme dry-weather flow averaged 308,000 gal. daily. Continuing, Mr. Roe stated:

The preliminary settling tank has removed an average of 43 per cent of the total solids. The aeration units have built up and maintained a typical activated sludge, but the final settling tanks have not given any better results than the average run of activated-sludge settling tanks in other cities. Some colloidal matter and finely divided floc has been noticed flowing over the final effluent weir at times, but evidently these light solids have little or no oxygen demand. The return run of the scrapers of the collecting equipment in these final tanks was raised to just below the flow line, instead of returning from the top at the inlet end of the tanks to the bottom at the effluent end, as has been customary practice. This change brought about the apparent reduction of a current at the bottom of the tanks and a slight reduction of floc going over the effluent weir.

Up to Nov. 1, 1928, four doses of well-digested and odorless sludge had been withdrawn and applied to the drying bed, and there reduced to a typical, well-dried and stable sludge. The digestion tank was not seeded with ripe sludge nor has any lime been added to the sludge. Odors are not perceptible near this tank, as it is covered, and the odors inside the building are not offensive. The slight scum on the liquor surface does not prevent the gas from escaping freely, due probably to agitation and hosing of the scum once or twice a week.

No laboratory is provided, but methyline blue stability tests have been made daily since January, 1928. Except for two periods of about five days each, every sample has stood up for at least ten days and most of the samples have been blue after twenty days. The first of the two unsatisfactory periods was caused by a large quantity of septic sludge, released from lodgment in the sewer system by an increase in flow; the second was caused by a quantity of sludge from the digestion tank overflow, the operator failing to close and open the proper inlet gate. Average results of 24-hour samples are tabulated below. The five-day biochemical oxygen demand on the final effluent was, in the order of the tests, 27.5, 12.25, 4.2 and 5.0. All results are in parts per million.

	Raw Sewage	Final Effluent
Suspended solids	43	19
Free ammonia and organic nitrogen	14.48	10.46
Nitrites and nitrates	2.48	4.19
Oxygen consumed	52.3	29.1

Quoting again from the paper:

With a sewage containing suspended solids in excess of 80 parts per million after May 1, the results improved, for as the quantity of sewage dropped, the concentration of activated sludge increased to and above the concentration found to be optimum. This permitted an excess to be sent to the digestion tank for the first time since the plant was started. Moreover, the condition of the sludge improved with the increased period of aeration. A complete daily record is kept by the operator and reported monthly to the state Department of Public Health. This covers rainfall, temperature, sewage flow (total and aerated), power consumption, quantity of screenings, sludge pumping, sludge return, settleable solids and stability.

The power used for the aerators averages 144 kw.-hr. per 24 hours, varying with the rate of sewage flow, since there is no way to raise and lower the aerators automatically as the quantity of sewage varies daily, nor is it practicable to vary their speed. Assuming that the quantity of sewage handled is equal to the capacity of the plant, or 500,000 gal. daily, the power requirement now is at the rate of 17.5 hp. per million gallons. The total average consumption at present for aerators, pumps and collecting apparatus is 220 kw.-hr. per 24 hours. The bar screen is cleaned four times a day, the screenings averaging ½ cu. ft. per day. The preliminary tank sludge removal has averaged 2,250 gal. per day.

A daily determination of the sludge concentration of the liquor in the aeration tanks is made by the usual one-hour retention in a graduate. The ratio of this concentration has been maintained close to 17 per cent. The excess activated sludge built up is discharged regularly to the digestion tank, along with all of the fresh solids from the preliminary tank. The quantity of activated-sludge liquor returned from the final settling tanks to the first aerating unit has been increased from 25 to 35 per cent of the total quantity of sewage, with slightly improved results.

The city employs a non-technical man as an operator and the designing engineers have been employed to supervise operation. The attention given by engineers of the Department of Public Health and by the manufacturers of the equipment has served to educate the operator. The average annual cost of operation will probably be about $3,480, as follows: power, 79,200 kw.-hr. at $0.025, $1,980; operator at $100 per month, $1,200; miscellaneous supplies, telephone, etc., $200; cleaning drying beds, $100. Assuming the average sewage flow for the year to be at the rate of 450,000 gal. daily, the cost of operation would be about $21 per million gallons treated.

GENERAL VIEW OF MORRISON CANYON SIPHON
Location, as well as relative amount of steel and concrete conduit, determined by studies of alternative designs.

Building High-Head Siphons on the Yakima Project

Low Leakage and Concrete Under Heads up to 153 Ft. Are Features—Total of 25 Siphons—Pressure Tunnel Head 523 Ft.

THE MAIN canal of the Kittitas division of the Yakima project required eleven inverted siphons in its 26-mile length and, including those built on the two branch canals as part of the construction program being carried out by the U. S. Bureau of Reclamation, a total of 25 siphons are being constructed. Where so many more or less similar structures were being built on the same job it was possible to develop methods and details that would simplify construction and at the same time insure a well-built conduit. A notable feature of the project is the 925 sec.-ft. pressure tunnel which is to convey water beneath the Yakima River under a head of 523 ft. In the following are described the methods of handling forms, making construction joints, anchoring barrels to piers and other items of siphon construction that were developed or improved as the work progressed. A description of the concrete canal lining on this project appeared in *Engineering News-Record*, Nov. 7, 1929, p. 722.

Because of the considerable size of the waterway required to convey 1,320 sec.-ft., the capacity for which the main canal was designed, and also because of the irregular topography in the upper part of the project, it was found desirable to use inverted siphons for getting the conduit across watercourses along the route rather than to continue on grade around them. The requirements were to have all structures 10 per cent oversize. The siphons are, for the most part, circular in section and in the main canal generally about 12 ft. in inside diameter, cast as monolithic reinforced concrete.

Near the upper end of the main canal, siphon heads are relatively low; hence the structures are correspondingly simple. Farther down the canal danger of leakage, head losses and accident to the structures increased with the higher heads; accordingly particular care was exercised in design and construction.

Where heads on the siphons are excessive and under certain foundation conditions steel pipe was substituted for the reinforced concrete. At Morrison Canyon siphon the barrel was changed from reinforced concrete to plate steel at a head of 108 ft., whereas at Horseshoe Canyon the change was made at 135-ft. head. On the north branch canal, Hayward Canyon siphon, constructed entirely of reinforced concrete, will operate under a 153-ft. head. On the south branch canals where the line crosses Taneum Creek the flow is carried for about 3,285 ft. across the valley in a 60- and a 54-in. precast concrete pipe on which there will be a maximum head of 207 ft.

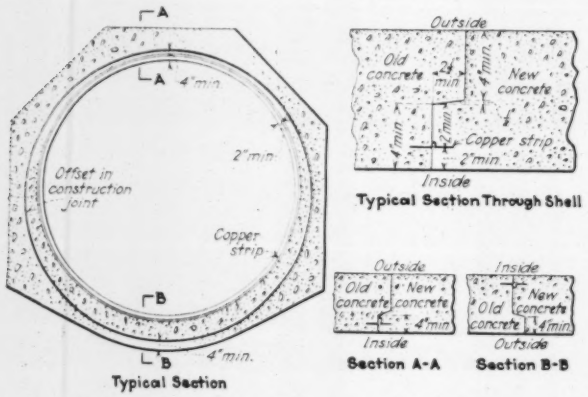
For siphon barrels, or for that portion of the barrel subject to heads of less than 100 ft., a 1:2:3.25 mix with 1½ per cent admixture of diatomaceous silica was used, whereas for barrels subject to heads in excess of 100 ft. a 1:1.7:2.8 mix was used without any admixture. By experiment it was found that the addition of diatomaceous silica to the leaner concretes resulted in a marked improvement in workability and impermeability and lessened the tendency for segregation. The addition of the admixture to the richer mixes did not prove beneficial. In no case was more than 3 per cent admixture used in any part of the project construction.

Construction of the barrel required both inner and outer forms. The inner form consisted of a series of circular ribs made up of 2-in. boards over which a facing of 2-in. tongue-and-groove beveled edge staves was placed. Thus the inner forms in place closely resembled a wood-stave pipe and in fact was built up in much the same manner. Each barrel form was continuous, no panel forms being used. The circular ribs, braces and lagging were used repeatedly.

Particular attention was given to joints between the successive pours, specially constructed joints with copper water stops being used between concrete placed on successive days. Concrete between construction joints was poured in one operation. Since the specifications required

that the barrel, for its entire periphery, be poured monolithically, it was necessary to provide construction joints to define each day's work. The specifications provide that these construction joints must be normal to the axis of the barrel, which required bulkhead forms to be built between the inner and the outer barrel forms on steep angles. A 30-ft. length of barrel was found to represent an average day's work for the concrete crews as employed on the siphons; hence the design provided for construction joints approximately 30 ft. apart. By making these joints with eccentric offsets, as shown in the accompanying sketch, the danger of cracks that would bypass the copper water stop was lessened.

It will be noted that as a result of the offset being made eccentric it is closest to the inside barrel form at the tops, midway between inside and outside forms at the horizontal axis, and farthest from the inside form at the bottom. This design gave, (1) at the top of the barrel, the maximum practical amount of pressure head produced by the fresh concrete to take up the possible shrinkage from below the sloping part of the offset, and the minimum amount of concrete below this part of the joint to shrink, and (2) at the bottom of the barrel, as broad a section of new concrete above the offset line as practicable, thus

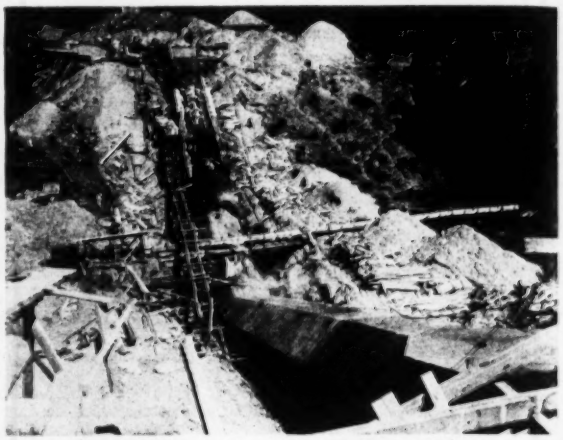


CONSTRUCTION JOINT DETAIL, SHOWING ECCENTRIC SHOULDER AND COPPER WATER STOP

increasing security against shearing off the projecting end of new concrete by such shrinkage as might occur in the concrete below the offset line. The weight of concrete from the sides above also aids in preventing such shearing action.

The copper water stop was placed inside of the reinforcing steel, great care being taken in riveting and soldering the joints, and repairing any damaged spots before pouring concrete. The copper strips were given a heavy coat of an emulsion of asphalt in water, which laboratory tests had showed to be effective in preventing leakage along the contact between concrete and copper. Expansion and contraction at the construction joints is provided for by a $\frac{1}{4}$-in. crimp rolled into the copper strips.

Because tie wires were not permitted to pass through the shell of concrete siphons, other means had to be found for fastening the inside form in place. This was accomplished by building concrete piers at about 8 ft. centers with the top of each at the elevation of the outside of the barrel shell at that point. In building these piers a steel bar was placed in each one with both ends embedded and with the exposed portion, about 4 ft. long, bent to fit the curve of the inner form. This bar, besides being used to support the inside form, constituted an

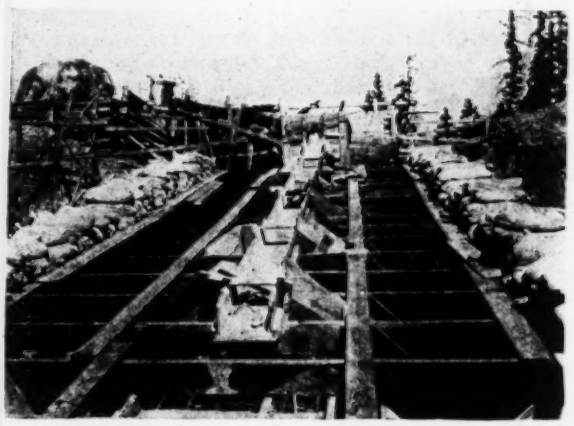


WORK UNDER WAY ON A TYPICAL CONCRETE SIPHON
All concrete siphons were covered by backfill where canyon slopes were not too steep.

anchor to which tie wires holding the inside form could be fastened and permitted concrete in the bottom of the barrel shell to be poured monolithically for its full thickness. As a further aid in preventing the form from floating when concrete was poured, cement sacks filled with earth were placed on top of the forms. Precast concrete blocks were placed along the top of the inside form to support the reinforcing steel. Wooden spreader blocks were placed between the inner and the outer forms above the spring line, but were removed as the pouring progressed.

The reinforcing steel in the barrel was lapped 40 diameters. Each transverse hoop consisted of two halves, the placing being such that laps on alternate bars would come at quarter points and on the bars between at three-quarter points, no laps being on the line of either vertical or horizontal diameters. In the higher head siphons two rows of transverse hoops were required, one row 3 in. from each face of the barrel. Splices in the longitudinal temperature bars were staggered so that the laps formed a complete spiral around the barrel.

After removal of the forms, which was in not less than fourteen days, the interior of the siphons was finished with carborundum bricks to remove prominent projections and fins wherever such occurred due to imperfection in the forms. This work was carried on from stagings held by cables and lowered into the siphon from



CLOSE VIEW OF OUTSIDE FORMS ON CONCRETE SIPHON
Concrete chuted to place. Bagged sand to prevent floating when concrete is poured.

either end as the work progressed. The result was a very smooth interior finish, and although no tests have been made, engineers on the project believe that for these concrete siphons the value of n in Kutter's formula should be not more than 0.014.

To avoid danger and loss that might result from leakage in the siphons, particular care was taken to keep leakage to a minimum. Tests on ten siphons gave leakages, shown in the accompanying table, which averaged only 0.0192 gal. per minute per foot of length. The exceptionally high loss in Dry Creek siphon was due to special conditions that should be taken into consideration. This siphon is a long, low-head structure built on a loose, coarse sand, gravel and boulder formation, and part of the work was done during summer months when the temperatures reached 110 deg. F. in the shade. In the ensuing winter there was a period of cold weather during which the temperature did not rise as high as the freezing point for nearly a month and for sixteen days went below zero every night, reaching a minimum of —22 deg. F.

The barrel of Dry Creek siphon was exposed during the cold weather and contraction cracks appeared in between some of the construction joints, which are 32 ft. apart. The test of this siphon was made early in the following spring before the contraction cracks had a chance to close as a result of warmer temperatures. The relatively high leakage, under these circumstances, was not considered serious. The foundation was in no danger of being softened or scoured away, and it is believed that after backfilling and under operating conditions the cracks will silt up.

The crossing of the Yakima River is to be made by means of an inverted siphon in tunnel having a total length of 3,649 ft. along the axis and operating under a maximum head of 523 ft. It will carry water for the north branch canal under the main transcontinental lines of the Northern Pacific Railroad and the Chicago, Milwaukee, St. Paul & Pacific Railroad, as well as under the Yakima River, Swauk Creek and a state highway. The pressure tunnel is circular in section with a diameter of 9 ft. 3 in. inside the concrete lining. At its lowest point the tunnel will be 142 ft. below normal water surface in the river. It will be equipped with an automatic wasteway consisting of a riser, at the upper end of which will be two 42-in. balanced needle valves capable of discharging the full capacity of the canal into the river. The needle valves will be electrically operated and controlled.

On Sept. 1, 1929, all of the main canal siphons and four of those on the north branch canal had been completed. On the south branch canal three siphons had been completed and construction of the others was in progress. At the Yakima River crossing the contractor had completed the excavation of the wasteway shaft and about 125 ft. of the tunnel, working both ways from the wasteway shaft and disposing of the tunnel muck through it.

The contract for building the main canal siphons was carried out by E. A. Webster & Company, of Spokane, Wash. The Utah Construction Company, of Ogden, Utah, has just completed its contract for four of the north branch canal siphons, and Morrison-Knudsen, of Boise, Idaho, has the contract for building those on the south branch canal. Charles and George K. Thompson, of Los Angeles, Calif., have the contract for building the Yakima River crossing. The work is being done under the supervision of Walker R. Young, construction engineer, U. S. Bureau of Reclamation.

INVERTED SIPHONS ON THE KITTITAS DIVISION, YAKIMA PROJECT

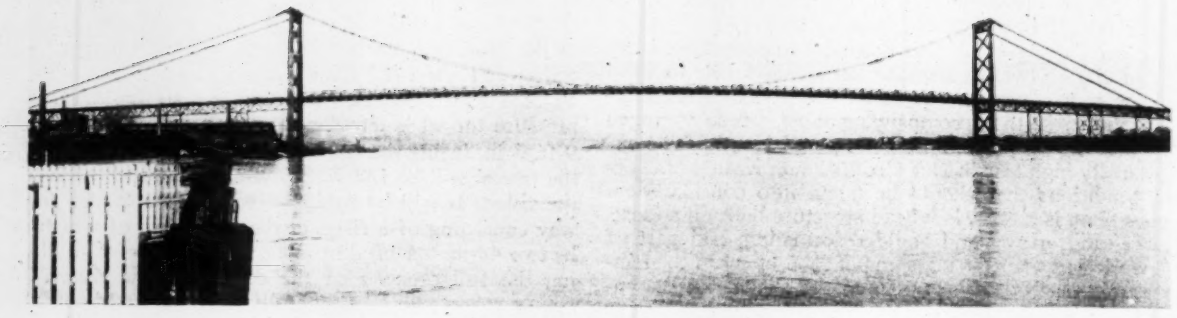
Name of Siphon	Inside Diam.	Max. Head, Feet	Length in Feet of Monolithic Concrete Pipe	Steel Pipe	Lock Joint Pipe	Max. Slope With Horizontal
Main Canal						
1. N. P. tunnel siphon (horseshoe section)	12' 3"	21 0	307			19° 30'
2. Milwaukee tunnel siphon (horseshoe section)	12' 3"	29 8	189			31° 00'
3. Tucker Creek siphon (square section)	11' 6"	12 2	75			19° 30'
4. Big Creek siphon (circular section)	11' 6"	24 7	105			19° 57'
5. Little Creek siphon (circular section)	11' 6"	24 7	105			19° 57'
6. Nelson Creek siphon (circular section)	12' 0"	53 7	372			17° 54'
7. Peterson Creek siphon (circular section)	13' 0"	70 8	282			45° 00'
8. Spexarth Creek siphon (circular section)	12' 0"	56 0	351			19° 40'
9. Cle Elum Creek siphon (circular section)	12' 0"	119 3	562			37° 20'
10. Horseshoe Canyon siphon (circular section)	12' 1"	135 5	675	151		33° 00'
11. Morrison Canyon siphon (circular section)	12' 1"	232 0	444	552		33° 57'
North Branch Canal						
12. Yakima River crossing (circular section)	9' 3"	523 0	3,649			55° 00'
13. Hayward Canyon siphon (circular section)	11' 2"	152 6	624			32° 30'
14. Sheep Dip Canyon siphon (circular section)	11' 2"	112 3	553			29° 15'
15. Dry Creek siphon (circular section)	11' 2"	47 1	1,551			9° 44'
16. Little Dry Creek siphon (circular section)	11' 2"	30 3	314			12° 00'
17. Johnson Creek siphon (lock joint pipe)	72"	109 4	259		1,530	23° 00'
18. Unnamed siphon (lock joint pipe)	72"	38 7	24		284	16° 30'
South Branch Canal						
19. Station 91 siphon (circular section)	70"	44 6	320			25° 51'
20. Taneum Creek siphon (lock joint pipe)	54"	149 3	18		1,212	30° 00'
	60"	207 1			2,072	
21. Rattlesnake Canyon siphon (lock joint pipe)	54"	71 2	18		425	26° 07'
22. Station 275 siphon (square section)	63"	11 0	93			15° 20'
23. Broadnax Canyon siphon (lock joint pipe)	54"	50 2	18		344	23° 30'
24. Robinson Canyon siphon (lock joint pipe)	60"	14 3	43		88	9° 16'
25. Manastash Creek siphon (lock joint pipe)	36"	19 6	8		409	25° 00'

RESULTS OF LEAKAGE TESTS ON TEN CONCRETE SIPHONS

Crossing	Length, Ft.	Max. Head, Ft.	Total Loss, G.P.M.	Loss, G.P.M. Per Lin.Ft., of Barrel
Little Creek	105	24 7	1/30	0 000316
Big Creek	105	24 7	1/30	0 000316
Nelson Creek	372	53 7	5½	0 014785
Spexarth Creek	351	56 0	⅛	0 000356
Cle Elum Creek	562	119 3	No loss in 24 hr.	
Peterson Creek	282	70 8	2½	0 009000
Sheep Dip Creek	553	112 3	11	0 019900
Hayward Canyon	624	151 6	4	0 006410
Dry Creek	1,551	47 1	170	0 109606
Little Dry Creek	314	30 3	9 7	0 030892
Average loss				0 019158

Germany Erecting First Skyscraper

The sixteen-story hospital now under construction at Schwabisch Hall, Germany, will be the first of the skyscraper type in that country and will be the largest and most modern hospital in Europe.



The Ambassador Bridge Crossing the Detroit River in an 1,850-Ft. Suspended Span

Detroit Suspension Bridge Opening Nine Months Ahead of Schedule

Remarkable Record Made in Dismantling and Re-erecting Structure With 1,850-Ft. Main Span—Some Changes in Design Necessary in the Interest of Speed

WHEN the decision was made last April to dismantle the Ambassador suspension bridge in Detroit and install new cables, the structure was well along toward completion and about one year ahead of schedule. So rapid has been the progress made in dismantling and re-erecting the structure that it will be opened as this is published, still nine months ahead of the scheduled opening date in August, 1930. This remarkable record is matched only by the similar work carried out on the Mount Hope suspension bridge in Rhode Island, in charge of the same contractors. Some of the methods used at Detroit are here set down as an indication of what a construction organization can accomplish when faced with the necessity.

The Ambassador bridge is a private toll structure connecting the city of Detroit with Windsor and the other border cities of Canada. It has a central span of 1,850 ft. and unloaded backstays. There are two cables, each consisting of 37 strands of 206 No. 6 galvanized wires. The roadway is 47 ft. wide, and there is a single 8-ft. sidewalk along the west side. Towers of unusual slenderness are utilized, and a departure of note in the fabrication is the attachment of gussets to the floorbeams in the shop, in order to facilitate erection. The design of the bridge was described in *Engineering News-Record*, Sept. 27, 1928, p. 460. Other articles have appeared, as follows: foundation construction, Dec. 6, 1928, p. 830; tower erection, Jan. 31, 1929, p. 186; decision to replace cables, April 4, 1929, p. 564.

At the time it was decided to take down the original cables, erection of the steelwork for the suspended span had been started and the first pass of the travelers was well under way. Steelwork was substantially in place for the American approach, and was completed on the Canadian approach. Subcontracts had been awarded for the paving of these portions of the work, as well as for paving the main span when it should be completed. Subcontracts also had been awarded for the construction of the two anchorage superstructures, concrete masses about 50x90 ft. in plan, rising up to the bridge deck level. The portions of these which incased the anchorage eyebars had been poured.

With the decision to remove the cables, work on the anchorage superstructures was stopped, the formwork taken down and stored and the eyebar incasement cut out, as described in *Engineering News-Record*, Oct. 10, 1929, p. 562. At the same time it was decided to concentrate on the approach paving, so that nothing would remain on completion of the new cables except steel erection and paving of the main span.

In order to carry this out, a change in the design of the anchorage superstructure was necessary. As originally designed, these superstructures were masonry piers which received the approach girder span on the one side and the deck truss span on the other side. Such a superstructure could not be built until the cables were spun, and if it were left uncompleted, it would be necessary to leave the truss span hanging as a cantilever and to leave the girder span on the ground. This meant that there could be no concrete deck on either of these two spans. In recalculating the time for completion it was found absolutely necessary to fill in this gap of pavement in order that the hauling of paving material across it to the main span could be commenced at once. Therefore, while the cables were being spun, steel bents were designed, fabricated and erected on the anchorage superstructure to receive the girder spans and the truss spans. With these in place, the pavement was completed.

In re-erecting the suspended span steel every effort was made to economize the time of the raising gang, and all materials possible were framed together on the ground or on the barge. The original plan of suspended-span design by which the floorbeams instead of the trusses were directly attached to the suspenders proved of great value in speeding up the work. Between Sept. 6 and Sept. 27 about 4,000 tons of structural steel was erected.

Erection of the suspended-span steel was effected by means of deck travelers, which at the first pass from the towers to the center of the span hung the floorbeams which carried the stiffening truss bottom-chord gussets

and then raised the bottom chords, bottom laterals and enough stringers to run on. On completion of this pass a day was devoted to backing the travelers empty to the towers. On the second pass the travelers placed the web verticals, web diagonals and most of the top chords of the stiffening trusses, as well as the sidewalk framing and sidewalk railings. On the final pass, backing from the center to the towers, the trusses were completed by inserting the remaining floor steel, erecting the roadway side railing and miscellaneous details.

By advancing the erection of the sidewalk railing and framing, it was possible to erect immediately on this framing a wooden roadway for transporting form lumber, reinforcement and concrete. All materials were hauled on this roadway by means of light gasoline tractors, and as soon as the travelers started back from the center on the final pass, it was possible to commence setting floor forms at mid-span, following the travelers to the towers. Thus, with the last steel being placed on Sept. 27, it was possible to commence pouring the roadway slab Oct. 7, and to complete it Oct. 15. In the eight days, 1,850 lin.ft. of 6-in. slab, 47 ft. wide, was placed. While this concrete was curing, preparatory to placing the asphalt surface, the road forms were removed and the sidewalk was poured.

While erection of the suspended-span steel has been under way, the anchorage superstructures have been carried up and are substantially complete. Final wrapping and painting of cables was commenced upon completion of the concrete slabs, and although these will not be completed when the bridge is opened to traffic, they will be completed shortly thereafter.

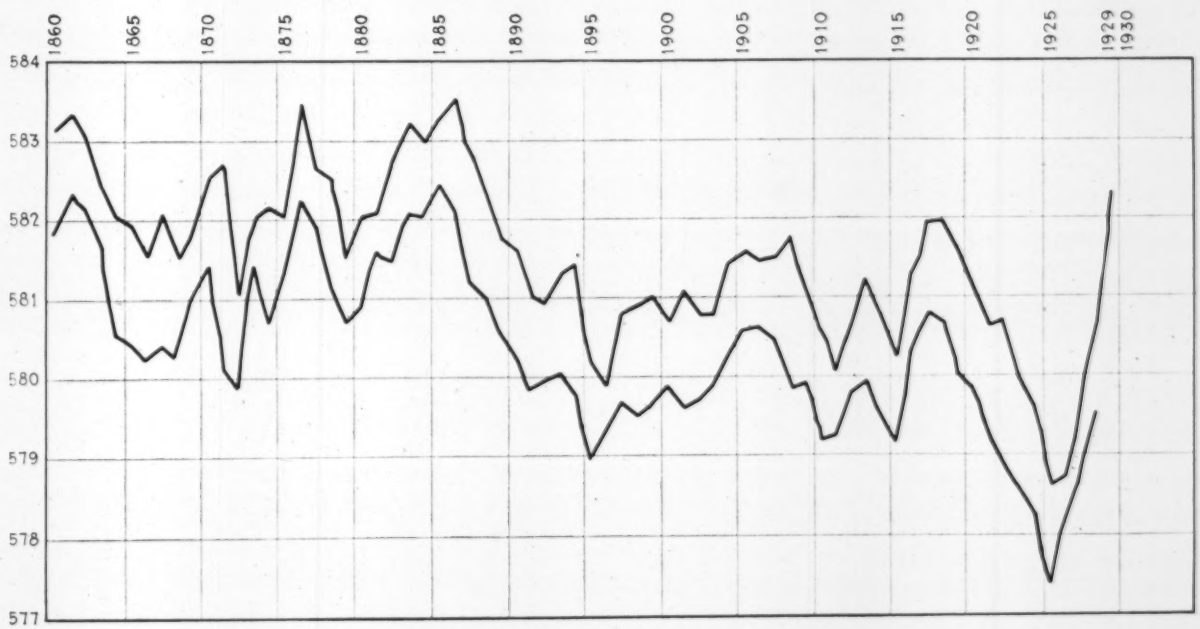
According to Jonathan Jones, chief engineer of the McClintic-Marshall Company, designer and erector of the bridge, complete credit goes to the field forces for this remarkable record. Robert MacMinn, assistant chief engineer and L. L. Martin, erection engineer, McClintic-Marshall Company, deserve credit for the way in which the operations were scheduled and pushed, and L. N. Gross, of the Keystone State Corporation, for the excellent record made in dismantling and re-erecting the cables. Mr. Jones also acknowledges the co-operation of the W. E. Wood Company, of Detroit, subcontractor for paving slabs and anchorage superstructures, and of the Zipp-Beckmeyer Company, of Huntington, W. Va., subcontractor for the electric lighting system.

Lake Levels and Damage to Shore Property at Chicago

Waterfront Development and Disregard of High-Water Stages Contribute to Damage by Recent Storms

In local accounts of the recent extensive storm damage to lakefront property, public and private, at Chicago, much stress is laid on the present high stage of Lake Michigan as a major cause. But a paper read recently before the Western Society of Engineers by W. G. Potter, drainage engineer of the Illinois State Waterways Division, shows that this lake elevation has been exceeded in previous years and that the destructive effects of the waves during the past few weeks may be attributed largely to the remarkable improvement developments along the lakefront during recent years, combined with a general tendency to insufficient consideration of the fact that high-water stages in the lake are of periodical recurrence. In regard to development and shore protection, this paper, of which an abstract follows, deals largely with private property. But it is evident that park and other authorities controlling public property along the lakefront must give more attention to the protection as well as to the development of such property. —EDITOR.

FOR the last few years Lake Michigan has been on a rampage. Protective piers, jetties and bulkheads have been destroyed. Concrete breakwaters have been ruined. Roads and boulevards have been undermined and put out



ELEVATION OF LAKE MICHIGAN, 1860 TO 1929
Showing maximum and minimum stages of monthly means.

of service. Trees, lawns and shorelands have been washed away or eroded, even along the high bluffs of the north shore. But let us look at the lake-level record.

Since 1860, a complete record of the lake levels has been kept and is shown in the hydrographs of the U. S. Lake Survey. This record shows the rise and fall of the lake surface by monthly means. In 1860 to 1863, in 1870 to 1871, in 1876 and again in the five-year period from 1883 to 1887 the present high stage at Lake Michigan was exceeded, the highest stage being in June, 1886, which was 1.24 ft. higher than our maximum in July of this year. Since 1887 the lake has been taking a middle course with the usual annual cycles until in January, 1926, it reached the lowest stage on record. From that time the stage has rapidly risen each year. This record of the levels of Lake Michigan from 1860 to date is shown in the accompanying drawing, which gives only the highest and lowest months of each year, intermediate months being omitted. The levels are taken from the hydrographs of the U. S. Lake Survey.

Of the three principal rises in lake levels, the first, 1872 to 1877, showed an increase from El. 579.87 to El. 583.49 in 43 months, or an average rate of 0.084 ft. per month. Its cause seems to have been excessive rainfall from 1870 to 1874. In the second rise, 1879 to 1887, the crest reached the highest point on record, and remained high longer than any other rise, but the main part of the rise lasted 41 months, with a rate of 0.062 ft. per month. Excessive rainfall was clearly the cause, as it was above normal for eleven years out of the twelve previous to the date of the crest height.

The present rise started from the lowest lake level on record, in January, 1926, and rose at the rate of 0.115 ft. per month (nearly double the other rates) until July of this year. Previous to the extreme low crest in 1926 the rainfall was below normal from 1917 to 1923 and since then it has been extremely heavy until this year. All of these rises cover about the same length of time from low point to crest. The outstanding feature is that while the first two rises reached a higher stage than the last one, still this last rise has a total rise and a rate of rising much greater than either of the others. On the other hand, part of the phenomenal present rise was due to the fact that the controlled flow from Lake Superior was considerably larger during the spring of 1927 than in previous years. With all of this in mind, it is the opinion of the writer that the crest in 1930 will be at least as high as the crest of this year, and may go a little higher.

While the water stage this year has been exceeded several times, still it is more than 40 years since the present level has been exceeded or even reached. In these 40 years Chicago and the north shore region have grown by leaps and bounds. Property along the lake shore which then was not settled and had no special value is now built up with great buildings, residences and parks. Boulevards, bathing beaches, pumping stations and other costly improvements line the lake shore. All of these have to be protected from the heavy winds and periodical storms.

During the years when the lake was low, the tendency of property owners was to build out on the beach as far as possible, raising the ground a few feet above the water and depending on low bulkheads or jetties for protection. As the lake rose, these bulkheads became less efficient and were either submerged or battered to pieces by the storms. For this reason, during the past year there has been a decided movement on the part of the property owners either to repair or to rebuild their shore protections.

Several types of shore protection in use come under two general classes: piers or jetties at right angles to the shore line, and bulkheads parallel to the shore. The object of the former is to stop the shore current, which is usually from the north, and thus cause the sand to drop and form an accretion. But in the course of a few years they are usually so damaged that their usefulness is practically destroyed. Also, with the rising lake level of the past few years, the waves have advanced so far westward that they cut around the shore end of the protection. In the same way, bulkheads are often too weak to withstand the storms and high lake level combined.

Shore protection has now become a necessity, and even should the lake start down again next year on a new cycle, the property owners should decide to have such protection carefully studied and designed. Most of the present methods are crude and temporary, especially where owners undertake individual work at the least expense and without reference to adjoining properties. Three recommendations are made by the writer: (1) The protection should be mainly by bulkheads or breakwaters parallel to the shore. (2) Timber bulkheads should be protected by steel sheeting of sufficient depth to prevent scouring or moving of the material behind the bulkhead. (3) With the increased value of the shore lands, a more permanent type of structure, of reinforced concrete, should be designed. Finally, property owners should combine and thus get a uniform and better construction at lower cost.

Yard Locomotives Dispatched by Radio

An experimental radio communication system for dispatching yard locomotives that has been installed at the West works plant of the Ashland division of the American Rolling Mill Company at Ashland, Ky., by the General Electric Company is being watched with interest.

Radio equipment for communication between the front and rear ends of long freight trains had been previously tried out by several railroad companies, but this is believed to be the first installment of its type in an industrial plant. The equipment installed consists of two 50-watt transmitters and power receivers; one set is installed in the dispatcher's office at the Hood's Creek scale house and the other is installed on oil-electric locomotive No. 742. The equipment operates on a frequency range of from 2,250 to 2,750 kilocycles and is set at 2,440 kilocycles, corresponding to a wave length of 122.9 meters. The transmitters have a broadcasting range of about 5 miles.

Two-way communication is provided which enables the dispatcher to issue orders and to receive confirming replies. The sets have sufficient volume so that the engineer can plainly hear the orders from the loud speaker above the noise of the train.

This system enables the dispatcher to issue orders directly to the train crew, eliminating chance of errors. Due to the flexibility of the system more work can be done with a given number of yard engines. Another great advantage of this system of control is that in weighing a train of cars in a heavy fog any number of cars up to the capacity of the engine can be handled. Under the usual system of control it is necessary to cut a train up into sections of eight or ten cars when weighing a train.

Experiment along this line will be continued and similar apparatus is now being installed by the Westinghouse Electric & Manufacturing Company for comparative purposes.

Fig. 1—Somerset Dam Sixteen Years After Completion

Long Period Settlement Records of Two Earth Dams

Assurance That Well-Compacted Earth Embankment Reaches Virtual Stability in a Few Years Given by Periodical Levels Taken for Sixteen Years on Somerset Dam and for Five Years on Davis Bridge Dam in Southern Vermont

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Measurements of progressive settlement in earth dams, during a long term of years, have been published so infrequently that the study presented in this article is distinctly unusual. Additional interest is given to these particular records also by the fact that they are secured from structures built by the semi-hydraulic process now so widely popular with earth-dam builders. Finally, the figures of settlement are made noteworthy by being accompanied by exact mechanical analyses of the materials which entered into the embankments. In interpreting the significance of the settlement charts and tables in which the results are summarized, the reader therefore is assisted by a complete background of the influencing conditions. —Editor.

RECORDS of settlement of two earth dams of considerable importance and magnitude completed on the Deerfield River in New England respectively sixteen and five years ago are given in the accompanying table and graph. These records are of unusual interest because they cover, in the case of one of the dams, a period longer than any that has come to the writer's attention, and because they show that both dams are approaching a condition of rest as far as settlement is concerned.

Somerset dam, on the east branch of the Deerfield River in southern Vermont, was completed and placed in service in 1913. It has a maximum height of 105 ft. above the stripped surface and was built by the semi-hydraulic method. The earth was excavated by steam shovel, conveyed to the site in cars and dumped from trestles into dikes, from which the finer materials were sluiced by water jets into a central pool to form the impervious core. Ten years after the completion of the Somerset dam another dam was built 20 miles below it on the main river, at Davis Bridge, Vermont. The Davis Bridge dam has a maximum height of 216 ft. above the bottom of the core trench.

The materials of both dams are boulder clay and glacial drift placed in nearly the same manner, the difference being that at Davis Bridge more of the material than at Somerset was dumped in the bottoms of the outer dikes before sluicing began. Also draglines were employed at Davis Bridge, fewer trestles were used and more sluicing was done.

The Deerfield River project, including Somerset dam, was described in *Engineering News*, Dec. 25, 1913, by a staff writer, and some details of the construction, sluicing and core were given by the present writer in *Engineering News*, June 4, 1914. The design and building of Davis Bridge dam were described in two articles in *Engineering News-Record*, Jan. 24 and Feb. 7, 1924, and by a third in the *Journal* of the Boston Society of Civil Engineers, Vol. 12, January, 1925. All three articles are by A. C. Eaton, hydraulic engineer, New England Power Company.

Mechanical Analysis of Dam Materials

Table I shows the mechanical analysis of the Somerset and Davis Bridge core and borrowpit materials. The analysis of the Somerset material was not carried beyond the 0.07 mm. size, sieve No. 200. Of this size, however, somewhat higher percentages passed 200 than of the Davis Bridge material, indicating that the Somerset core contains as much of the finer sizes as were found at Davis Bridge. Fig. 2 shows the typical uniformity curves of a fine clay, together with the diameter in millimeters of the particles composing it. To show the characteristic difference between clays and sands, curves *A* and *A1* have been introduced into the figure.

Fineness of sand, unless colloidal sizes accompany it,

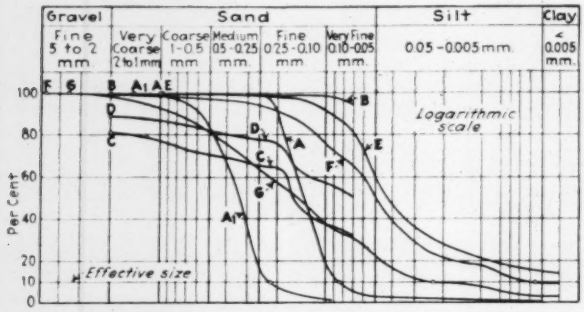


FIG. 2—UNIFORMITY CURVES DEVELOPED FROM ANALYSIS OF MATERIALS FROM DAVIS BRIDGE AND SOMERSET DAMS

A—Fine white quartz sand from bottom of core trench, Scituate dam, Providence (R. I.) water supply. *A1*—A hill sand from near Muskegon, Mich. *B*—Core material from Somerset dam, sample No. 1. *C*—Borrowpit material from Somerset dam, sample No. 2. *D*—Sample No. 3. *E*—Sample No. 1. *F*—Sample No. 2, core material from Davis Bridge dam. *G*—Borrowpit material from Davis Bridge dam, sample No. 3. See footnote Table I.

does not insure a low seepage rate; on the contrary, the rate may be exceedingly high. The sand represented by curve *A* was rejected for core purposes because of its high seepage rate. Its effective size was but 0.082 mm.; its uniformity coefficient, however, was very low, 1.8. Seventy-three per cent of the sample had a range in diameter of but 0.1 mm. Its seepage rate (maximum rate) was from 387,000 to 514,000 gal. per acre per day. There was no loss by ignition, indicating that there was little or no colloidal content. The uniformity coefficient of the curves *F* (core material) and *G* (borrowpit material) are respectively 11.3 and 9.6.

Record of Settlement

The data given in Tables II and III and the curves, Fig. 3, were derived from the records of elevations taken on benchmarks at the respective dams. The benchmarks at Somerset are on boulders on and near the top of the dam, those at Davis Bridge are on 2-in. pipes driven into the top of the dam. Reference benchmarks are located on monuments and ledges beyond the ends of the dams and away from the influence of settlement of the structures.

TABLE I MECHANICAL ANALYSIS OF SOMERSET AND DAVIS BRIDGE DAM BORROWPIT AND CORE MATERIALS

Diam. in Mm. Passing Sieve	Sieve Number Equals Wires Per Lin. In.	Type Names of Material	Per Cent by Weight Passing — Somerset, Sample No. 2 No. 1 Core *B*	No. 3 Borrowpit *C*	No. 3 Borrowpit *D*	Davis Bridge, Sample No. 1 Core *E*	No. 2 Core *F*	No. 3 Borrowpit *G*
4.71	4	Fine gravel					100.0	
3.50	6		(0.06)	(18.6)	(11.5)		99.2	100.0
2.02	10		99.94	81.4	88.5		98.8	99.8
0.97	20	Very coarse sand	99.92	76.9	86.7	100.0	98.2	92.6
0.63	30	Coarse sand	99.90	71.9	83.7			
0.44	40	Medium sand	99.89	69.7	81.4	99.6	97.2	78.2
0.25	60		99.88	65.3	77.7	98.4	93.8	63.4
0.20	80		99.88	63.9	75.7			
0.15	100	Fine sand	99.50	47.5	63.5	97.0	87.0	50.4
0.10	150					91.0	74.2	37.0
0.07	200	Very fine sand	93.98	32.0	50.3	82.0	65.2	31.0
0.05						76.6	61.0	28.0
0.04						46.5	39.2	16.8
0.022						29.3	22.9	9.4
0.012		Silt				21.5	18.5	8.1
0.008						17.6	12.4	4.6
0.006						15.7	10.2	3.5
0.004		Clay (contains colloidal sizes)				14.5	9.1	3.0

Of the Somerset samples, No. 1 is the average of two analyses of core material loss by ignition, 2.15 per cent. No. 2 is the average of two analyses of west borrowpit material; loss by ignition, 1.05 per cent. No. 3 is the average of two analyses of east borrowpit material; loss by ignition, 1.61 per cent. The figures in parentheses in columns 1, 2 and 3 are percentages retained on No. 10 sieve.

Of the Davis Bridge samples, No. 1 was taken from the center of the core at El. 1,287. No. 2 is composed of equal parts of material taken at 30 ft. and 60 ft. upstream from the center line at El. 1,289. No. 3 is a composite sample taken from six different borrowpits with coarser particles screened out.

At each dam the first elevations for records of settlement were taken about eight months after completion, therefore the amount and rate of settlement that took place in these first eight months are not known.

The percentages tabulated in the individual columns were determined by dividing the total settlement to that date by the depth of fill at the particular benchmark and reducing to percentage of 1 per cent of depth. The percentages thus obtained are directly comparable and are the data used in plotting the curves, Fig. 3. It will be noted that in the columns of Table II the percentages are much less uniform than in Table III. The uniform results in Table III are as they should be and are the result of the type of benchmark used at Davis Bridge—that is, pipes driven into the dam crest.

At Somerset the benches are on boulders more or less embedded in the earthfill and subject to movement by

TABLE II—PER CENT SETTLEMENT, SOMERSET DAM

Bench Elevations, Determined		July 15, 1914	Oct. 8, 1914	May 25, 1916	May 19, 1920	July 5, 1922	July 10, 1928	May 31, 1929
Elapsed Time, Years			0.24	1.87	5.85	7.98	13.89	14.89
Bench-marks	Depth, Ft.	Settlement—Percentage of 1 Per Cent of Depth of Fill						
1	23	Dam completed Nov. 29, 1913. First bench elevations taken July 15, 1914.	0.09	0.04	0.35	0.35	0.39	0.48
2	19		0.16		1.32	1.32	2.32	2.63
2a	41		0.15	0.54	0.85	0.88	1.12	1.20
3	57		0.09		0.58	0.72	1.23	1.26
4	60		0.13	0.17	0.38	0.47	0.53	
5	63		0.14		0.46	0.41	0.54	0.60
6	60		0.25	0.58	0.80	0.75	0.97	0.98
7	68		0.10	0.31	0.49	0.54	0.69	0.78
8	73		0.12	0.40	0.58	0.67	0.86	0.92
9	60			0.12	0.40	0.48	0.67	0.73
10	83		0.10	0.35	0.61	0.69	0.83	0.92
11	100		0.13	0.49	0.65	0.74	0.83	0.87
12	91		0.08	0.32	0.49	0.55	0.65	0.76
13	59		0.65	0.39	0.56	0.63	0.71	0.80
Average, omitting bench marks 2, 2a and 3			0.12	0.32	0.52	0.57	0.70	0.78

TABLE III—PER CENT SETTLEMENT—DAVIS BRIDGE DAM

Bench Elevations Determined		Aug. 13, 1924	Sept. 26, 1924	Nov. 11, 1924	June 2, 1925	Dec. 2, 1925	May 14, 1926	Oct. 26, 1926	July 11, 1928	June 1, 1929
Elapsed Time, Years			0.12	0.25	0.81	1.31	1.75	2.21	3.92	4.81
Bench-marks, Sta.	Depth, Ft.	Settlement—Percentage of 1 Per Cent of Depth of Fill								
3	44	Dam completed mid-December, 1923. First bench elevations taken Aug. 13, 1924.	0.00	0.05		No record			0.10	0.01
4	58		0.00	0.05			0.01	0.07	0.12	0.22	0.27
5	78		0.01	0.05	0.01	0.03	0.10	0.15	0.24	0.27
6	96		0.01	0.05	0.03	0.05	0.12	0.16	0.25	0.27
7	121		0.00	0.03	0.02	0.04	0.12	0.15	0.24	0.27
8	143		0.00	0.04	0.03	0.07	0.14	0.18	0.27	0.33
9	192		0.01	0.04	0.05	0.08	0.15	0.18	0.25	0.30
10	216		0.01	0.03	0.06	0.08	0.15	0.17	0.23	0.27
11	175		0.02	0.04	0.06	0.09	0.16	0.19	0.25	0.29
12	105		0.04	0.08	0.05	0.08	0.16	0.19	0.28	0.33
13	57		0.03	0.06		0.02	0.10	0.13	0.23	0.35
Average omitting bench-mark 3			0.01	0.05	0.04	0.06	0.13	0.16	0.25	0.30

frost and settlement not related to dam settlement. Other differences are due to wind at time of taking elevations and to instrumental errors. One or both of these latter may have produced the plus elevations obtained at Davis Bridge June 2, 1925, and effected the elevations of Dec. 2, 1925, the combination of which caused the hump in the Davis Bridge curves in 1925. The drop at the end of the Somerset curve in 1929 (Fig. 3) may have been due to the same causes.

The question might be asked, Why, for the same length of time, do the Somerset settlements show a greater amount than do these at Davis Bridge? The reason may be that the boulders on which the benchmarks are located have settled into the fill. This would account for the unevenness of settlement at individual benches at Somerset, but would not change the general form of the settlement curve. Three benches at Somerset, *2*, *2a* and *3*, have quite evidently settled or moved down the slope, due to their location and exposure on the downstream side of the dam. The records of

these three benchmarks have been omitted. The records of June 2 and Dec. 2, 1925, at Davis Bridge are erratic and out of line. Probably the dashed line under the hump in the Davis Bridge curves gives more nearly the actual conditions than those shown by the June and December records.

Actual settlement in feet during the time records have been made—that is, between July 15, 1914, and May 31, 1929—for Somerset dam was: minimum, 0.11 ft. at benchmark 1, depth of fill 23 ft.; maximum, 0.87 ft. at benchmark 11, depth of fill 100 ft.; and for Davis Bridge, Aug. 13, 1924, to June 1, 1929, minimum, 0.155 ft. at benchmark 4, depth of fill 58 ft.; maximum 0.58 ft. at benchmark 10, depth of fill 216 ft.

Rate of Settlement Curves

Plotting the results obtained in Tables II and III on a time per cent of depth scale, the curves shown in Fig. 3 are obtained. Three curves have been plotted for each dam, one of average settlement for the entire dam, one of settlement at the maximum depth of fill

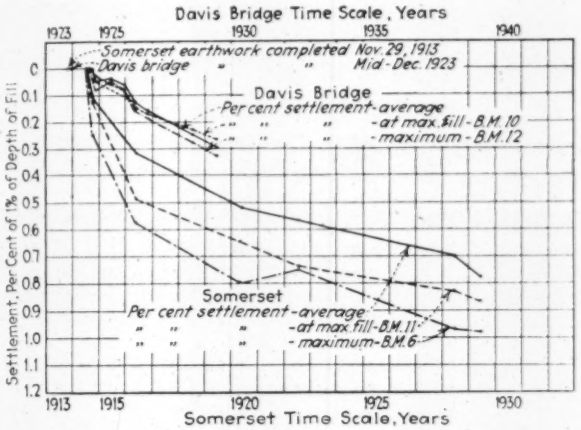


FIG. 3—RATES OF SETTLEMENT OF EARTHFILL, DAVIS BRIDGE AND SOMERSET DAMS

and one at the benchmark of maximum rate. The general form of these curves discloses the interesting information that the dams are already approaching a cessation of settlement. This is apparent in the sixteen-year record of the Somerset dam and is being followed in the shorter record at Davis Bridge. Especially is the similarity shown in the paralleling of the two sets of curves beginning 2½ years after completion of the individual dams. The value of this information is important and reassuring to designers and builders of earth dams. That the magnitude of settlement, within close limits, at Somerset and Davis Bridge may be applied in the design of earth dams to be built under other methods and with other material may not be assumed, but it would appear that it may be assumed that a well-compacted earth structure will reach a condition of comparative stability within a reasonably short period of time.

In placing an earthfill to support a large outfall sewer between Squantum and Moon Island in Boston harbor previously to 1890, the Boston Main Drainage Board found it advisable to delay the construction of the masonry sewer until such time as the supporting fill had reached a reasonably satisfactory state of settlement. To aid in determining when this condition had been reached,

FIG. 4—DOWNSTREAM SLOPE OF SOMERSET DAM MAY 31, 1929

a number of metal plates, with rods attached, were embedded in the fill. The elevations of the plates were recorded over a period of nine years, and from these data a curve of rate of settlement and a formula were derived.

By means of the formula the time when the settlement would reach a safe rate, in this case 0.01 ft. per year, was determined and the settlement that would take place in that time was provided for by giving the sewer invert the superelevation required. Superelevations ranged from 0.04 to 0.21 ft. The length of time for the maximum superelevation to settle to grade was estimated to be ten years. During construction, rods were placed in the top of the masonry over the points were the original plates and rods had been located. Elevations were taken on these rods from time to time after the completion of the sewer.

Satisfactory performance resulted. The recording of elevations was continued until no further settlement took place, proving that the formula derived from previous settlement was applicable to the conditions there existing in fill and underlying materials. However, the formula derived for the Moon Island fill was found not applicable under the conditions that obtained in respect to the materials and methods that were employed at Somerset and Davis Bridge.

Present Conditions at Dams

The present freeboard (height of dam crest above spillway crest) at Somerset is 9.5 ft. and at Davis Bridge 15 ft. Both of these are greater than the designed freeboard. Full reservoir (spillway level) at Somerset is El. 443 and at Davis Bridge El. 1,386 above mean sea level. Fig. 1 is a view of Somerset dam looking east

FIG. 5—MATERIAL RELEASED BY DECAYED TRESTLE MOVING DOWN SLOPE, SOMMERSET DAM

from near its northwest end and showing the outline of its downstream slope; and also trees, poplars, that have become seeded by the wind on the upstream slope along the water's edge and allowed to grow. The keeper's house is in the right background and for scale the engineer's auto may be seen on the crest of the dam. Fig. 4 shows the southwest slope of the dam. The slopes and benches formed by dumping from trestles were not completely graded when the dam was built but have assumed stable slopes and have become grass- and weed-grown. Some trees, mainly poplars, and shrubs have been allowed to remain, but the slopes have been kept free or large growth.

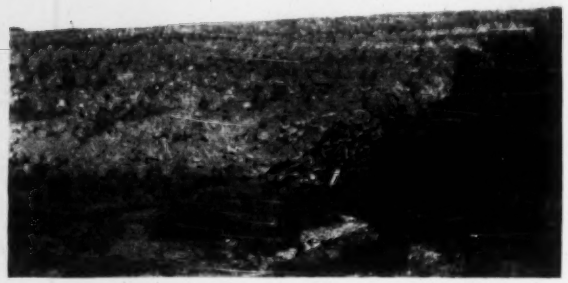


FIG. 6—DOWNSTREAM SLOPE OF DAVIS BRIDGE DAM

In completing the ends of the dam at and near the crest, the fill was widened to make greater surface contact with the hillsides. This was accomplished by moving the trestles out at the ends of the dam; the result was a wider crest and steeper side slopes at this point. So long as the trestle bents remained sound, they supported these steeper slopes. The trestle timber, portions of which are seen in Fig. 5, has decayed and recently the slopes have moved. This small area of movement is confined to the top of the dam and is adjusting itself to the new conditions. There are no other areas of movement on the dam. As at all earth dams abutting against pervious hillsides, there is seepage from the hills at the ends of the dam. This seepage has always been small at Somerset and there is no seepage through the dam.

Fig. 6 is a view of the downstream slope of Davis Bridge dam, showing its full 200 ft. of height from riverbed to crest of dam. The flatter slopes at the toe of the dam, indicated by dark areas, have become somewhat gullied and further flattened by the flow of surface water from the large area of the dam. The dark lines, parallel with the crest, are rows of evergreen and deciduous trees that have been planted on the slope. The area has been seeded to grass and clover, but is still subject to erosion where the slope has not become stable. As at Somerset, there is seepage through the hills at the ends of the dam. In the matter of permitting trees to grow on an earth dam, the result, when the growths have become large and old, has been detrimental in most instances.

The writer was resident engineer at Somerset. Through the courtesy of the New England Power Company, he recently visited this dam, sixteen years after its completion, also Davis Bridge and the other hydroelectric developments along the Deerfield. The company's engineers have kindly furnished the principal data on which this article is based.

Many Applications Filed for Boulder Dam Power

REQUESTS placed with the Secretary of the Interior for power from the Boulder dam project aggregate far more power than can possibly be generated, as shown by the following list:

Nevada-Colorado River Development Commission, Carson City, Nev.:
Three applications, one for 180,000 hp., one for 25 per cent of total amount developed and one for one-third of total amount developed. Assumed load factor, 76.5. Kilowatt-hours per year, 1,200,000,000.*

Metropolitan Water District, Los Angeles:
All or equitable allocation (280,000 hp.). Assumed load factor, 92. Kilowatt-hours per year, 1,683,000,000.

City of Los Angeles:
520,000 hp. Assumed load factor, 55. Kilowatt-hours per year, 1,869,000,000.

Southern California Edison Company, Los Angeles:
One application for all or equitable allocation and another for 62.3 per cent of total power available at Boulder dam, or single purchasing agent for all power for southern California. Kilowatt-hours per year, 2,243,000,000.*

Southern Sierras Power Company, Denver:
One application for 45,000 kw. and another for 7.94 per cent of all power generated. Kilowatt-hours per year, 286,000,000.*

Los Angeles Gas & Electric Corporation, Los Angeles:
One application for 45,000 kw. and another for 73,000 hp., or one-third of the amount allotted to California. Assumed load factor, 55. Kilowatt-hours per year, 262,400,000.†

San Diego Consolidated Gas & Electric Company, San Diego:
3.9 per cent of total allocated to southern California. Kilowatt-hours per year, 105,300,000.‡

Palo Verde, Mesa and Chuckwalla Valley Development Association, Los Angeles:
18,000 kw.

City of Santa Ana, Calif.:
10,000 hp. Assumed load factor, 45. Kilowatt-hours per year, 29,407,000.

J. T. Dobbins, Fredonia, Ariz.:
Amount not stated.

Katherine Midway Mining Company, Los Angeles:
5,000 firm hp.

City of Glendale, Calif.:
50,000,000 kw.-hr. Assumed load factor, 45.

United Verde Copper Company, Clarkdale, Ariz.:
20,000 kw. to 100,000 kw. Assumed load factor, 92. Kilowatt-hours per year, 161,184,000.

City of Pasadena, Calif.:
24,500 firm hp. Assumed load factor, 45. Kilowatt-hours per year, 72,051,000.

City of Burbank, Calif.:
20,000,000 kw.-hr. per year. Assumed load factor, 45.

Consolidated Feldspar Corporation, Trenton, N. J.:
120 hp. to 325 hp.

Yuma Utilities Company, Denver, Colo.:
20,000 kw. Assumed load factor, 55. Kilowatt-hours per year, 96,360,000.

Public Utilities Consolidated Corporation, Kingman, Ariz.:
100,000 kw. Assumed load factor, 45. Kilowatt-hours per year, 394,200,000.

Mohave County, Ariz.:
100,000 hp.

City of Newport Beach, Calif.:
10,000 hp. Assumed load factor, 45. Kilowatt-hours per year, 29,407,000.

Arizona Power Company, St. Louis, Mo.:
30,000 hp. Assumed load factor, 75. Kilowatt-hours per year, 147,037,000.

State of Utah, Colorado River Commission:
50,000 hp. Assumed load factor, 75. Kilowatt-hours per year, 24,060,000.

City of San Diego, Calif.:
28,000 hp. Assumed load factor, 100. Kilowatt-hours per year, 200,000,000.*

Details of a plan of apportionment proposed by Secretary Wilbur were given in *Engineering News-Record*, Oct. 24, 1929, p. 670. A further hearing on the subject

*Based on 3,600,000,000 kw.-hr. †73,000 hp. used. ‡Based on 2,700,000,000 kw.-hr.

was held in Washington Nov. 12, when the various interests argued for further modifications of the plan.

California Aqueduct

Estimates now available indicate that the aqueduct, to bring Colorado River water into southern California for domestic purposes, will cost in excess of $200,000,000, while $20,000,000 additional will be required for the proportionate share in the construction of the power plant at the dam and $30,000,000 will be necessary for transmission lines. The eleven cities making up the Metropolitan Water District will bear this expense on a basis of their assessed valuation, according to present plans. The city of Los Angeles under this program would be expected to pay 82 per cent of the cost, while Pasadena, the second wealthiest unit in the district, will pay 5.46 per cent of the total cost, based on the county assessment.

Before bond issues can be called for in the eleven respective cities it will be necessary for the Metropolitan Water District to obtain further information concerning an aqueduct route. The engineers of the district are expected to make their report on routes in November, and then a board of review will study their recommendations for several months, seeking to obtain detailed information beyond that submitted by the engineers.

Inferior Structural Steelwork

Some Details That Were Not Designed for Publication

By A. W. A. Eden
East Orange, N. J.

INSPECTION of certain structural steelwork, both at the shops and in buildings under construction, has created in the writer a much greater confidence in steel than had existed previously. If some of the advocates of higher unit stresses could see the kind of workmanship and the second-hand steel that is used in non-

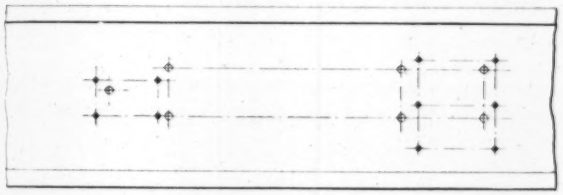


FIG. 1—EXAMPLE OF HOLES IN WEB OF SECOND-HAND BEAM

fireproof apartment houses, both speculative and otherwise, they probably would revamp their ideas, or else take steps to devise some method to improve the exceedingly undesirable conditions now existing.

Much of the second-hand steel is in excellent condition, with comparatively few old holes, and these in locations where there is no objection to them. On the other hand, the rust, the dirt and the many various-sized holes that occur in other pieces make much of this steel very undesirable and often objectionable. This is especially the case where many web holes exist in a carrying beam or girder and new holes have to be provided, some of which may be very close to the old holes—often much closer than good practice would sanction.

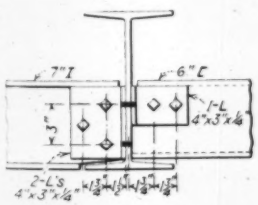


FIG. 2—INADEQUATE CLIP CONNECTIONS
If bolts at left fail, "it only means that web of 7-in. I will come to rest on the flange of carrying beam."

It is well known that the oxyacetylene torch has been and is being very much abused in fabricated steelwork, but it will be a surprise to some engineers—doubtless especially so to bridge engineers—to know that there are many instances where holes have been burned for *all* connections except those in small angle and plate fittings. Even holes in the thin webs of beams, channels and H-sections have been made with the torch, notwithstanding the ease with which they may be punched, for the simple reason that it is easier to move a burning outfit than to take the member to a punching machine. Not only field connections but shop connections also (eccentric and otherwise) have been bolted to a very great extent. Holes for ¾-in. bolts have been found on 1¾-in. centers where a 4-in. angle is used for framing. The writer has seen cases where small channels with one end wall bearing were expected to be framed with one 4x3x¼-in. angle, obviously the field connection to be made with only one bolt. The contractor says, "The channel cannot turn after the other end is built into the wall." What argument! What appalling evidence of fearfully deficient knowledge of the principles of sound engineering!

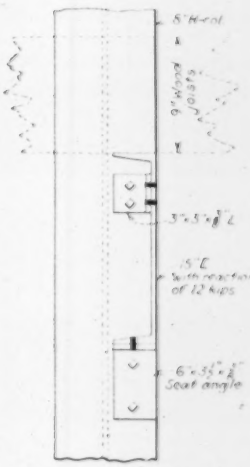


FIG. 3—SEAT ANGLE AND SIDE CLIPS CARRY 12,000 LB.

Similar argument is advanced to excuse a shortage in the required number of bolts in some connections—for instance, beams framed together with bottom flanges flush. Suppose, they say, that the bolts should partly fail; the web of the coped beam will come to a bearing on the flange of the carrying beam and "it can't get away"! Frequently one-sided connections receive no special consideration and no additional bolts are supplied to allow for eccentricity. Some shops have provided connections to columns assuming part of the load as carried by a seat angle and the remainder by a side clip—all bolted. Angles 6x3½x$\frac{5}{16}$ in. in size have been used, not only for framing connections but also at column bases and for beam seats. Government anchors are made of any material that is handy—perhaps ⅜-in. square reinforcing bars. Frequently no holes are provided for these anchors and sometimes the mason forgets to insert them for one reason or another. Bear-

ing plates are frequently only ¼ in. thick, sometimes scarcely that, no consideration being given to bending on account of projection beyond beam or channel flanges.

Granted that under wide-flanged beams and girders these plates merely serve the mason in giving a finished height, the cases of channels and narrow-flanged beams are very different. The usual 8 in. of bearing on a 12-in. brick wall is sometimes reduced to little more than 4 in., making it most desirable to have a plate of adequate thickness as well as of sufficient area to distribute the reaction, especially since the quality of the mortar may not always be equal to specification or building code requirements.

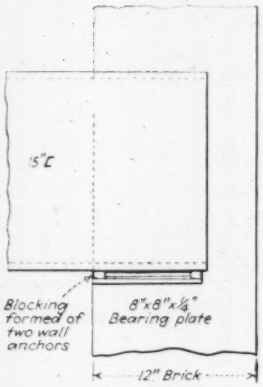


FIG. 4—BEARING PLATES ¼-IN. THICK SOMETIMES CONSIDERED ADEQUATE

Pipe separators are sometimes sheared and partly restored to cylindrical form with a hammer. Still it must be admitted that they serve fairly well the purpose for which they were intended.

Perhaps the most flagrant violation of the rules of good practice, common sense and building codes is the absence of facing for the bearing ends of columns as fabricated by the small shops, many of which seem to know nothing about milling machines or even "Newton" saws. Column material usually comes to the shops after having been cut to length with a friction saw. These cuts are fairly square but often not at right angles to the center lines of the full-length section. At the shops a small try-square is used in an attempt to detect any outstanding deviation from a square surface, hammers being brought into play to secure the desired finish. But frequently there will be conspicuous burrs, the removal of which has not even been considered or attempted. When such columns are erected we may have a fair bearing over 65 to 75 per cent of the cross-section, but some joints will be less than 50 per cent in actual contact bearing. Some bad joints occur where a section has been cut by means of the torch and smoothed as much as possible by the hammering process. Naturally the bearing surface in such cases is far from 100 per cent and our allowable unit stresses may be greatly exceeded in many instances. Wedging after completion of the erection seems to be the only remedy for such defects, but this is usually far from satisfactory in providing a good bearing.

Some fabricators seem to think that a base plate 16x16x¾ in. will carry any load, whether used under a 6-, 8- or 10-in. section. Drawings may give definite sizes for these plates, but since there is no supervision by the architect and inspections are made only occasionally by building department representatives and loan companies, material may be in place before it is discovered that such information has been ignored. Perhaps a plate specified as 24x24x1½ in. may be changed to two plates 22x22x¾ in. on the assumption that the total thickness is correct and that the plate should not extend to the edges of the cap plate on the brick pier. The thickness of pier cap plates as supplied is often inadequate for the distribution of the column and girder loads. Occasionally they are specified very carefully, but no size given for the column base plate.

The general assumption seems to be made that this kind of construction has been going on for years with no failures reported and that therefore it is altogether excusable, and no improvement is necessary. The question arises: "What can be done to remedy these conditions?" The following suggestions are made:

1. Insistence by building department examiners that filed plans give complete information as to size and thickness of column base plates and of cap plates of brick piers; also sizes of special bearing plates for heavy girder reactions, and details of unusual connections.

2. More careful inspection before erection of the steel delivered at the building site, and rejection and removal of all materials that is unsatisfactory, including all columns the joints of which are not faced.

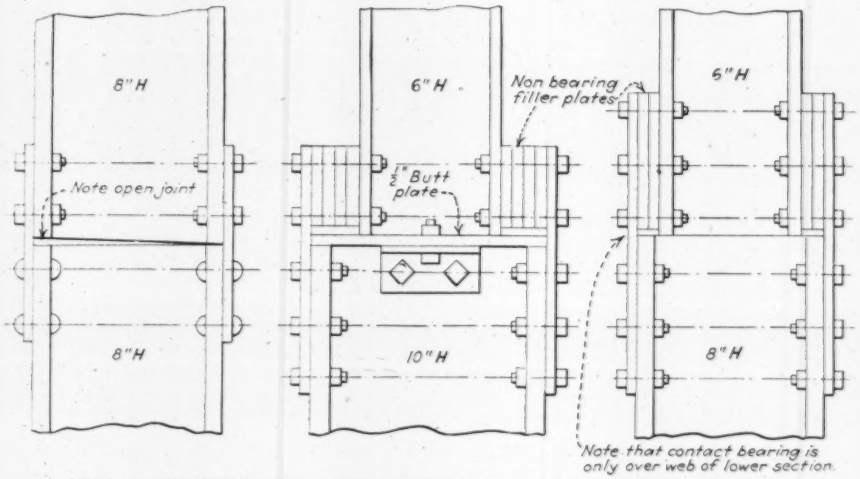


FIG. 5—EXAMPLES OF COLUMN SPLICES THAT WERE CONSIDERED "GOOD ENOUGH"

3. Compulsory shop riveting to the columns of all seat angles. This obviates the common practice of removing bolts on the job and more easily provides the necessary strength, especially under eccentric loading. Bolts would be allowed for all field connections, but caution must be used to insure a sufficient number where they are in bearing on the thin webs of beams and girders. This applies to shop bolts of framing angles and also to field bolts of connections coming opposite on girder webs. Customary minimum spacing of holes should not be decreased, neither should minimum distances to sheared edges, nor to rolled edges. No connection should be made with less than two bolts, except perhaps in a case where a member is merely to be held in position.

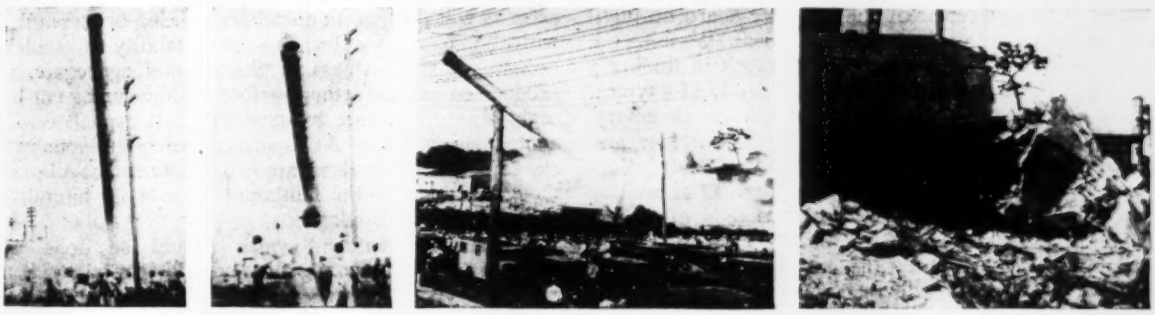
4. Licensing of fabricating shops by the state. This would seem just as reasonable as licensing architects and engineers, since the designs should be carried out in a thoroughly workmanlike manner by competent workmen in well-equipped plants.

High Concrete Stack Razed in Close Quarters by Blasting

Careful Shooting Topples 166-Ft. Structure Within Desired Limits, Progress Conforming Exactly to Predetermined Schedule

By Emil Nordstrom
Consulting Engineer, St. Petersburg, Fla.

SURROUNDED by a creek, two main streets and a railroad, a 166-ft. reinforced-concrete stack of the Florida Power Corporation at St. Petersburg was successfully razed and dropped within a narrow required space by blasting. Demolishing the old stack became necessary when the power company abandoned the old plant, which had become obsolete. The stack, 12 ft. 8 in. in diameter at the base, was so situated that two of the main streets of the city were within reach of its fall, First Ave. North being on the south side, and Sixteenth St. on the west side. The right-of-way of the Atlantic Coast Line Railroad ran within 60 ft. on the east side.



HIGH CONCRETE STACK RAZED BY BLASTING

To clear the street, the railroad and Booker Creek on the north side, it was necessary to drop the stack in a northwesterly direction with a differential of 20 ft. for the top of the stack after the fall. No records of the construction or details of reinforcing in the stack, which was built in 1913, were available.

Because of the necessity of dropping the stack within close limits, it was necessary to put a cable at the top pulling in the required direction of fall. The method used by a steeplejack in fastening this cable to the top of the stack proved interesting. There was no ladder either inside or outside of the stack and the only access to the top of the structure was by means of a copper wire serving as a ground for lightning rods. A series of wooden strips 1x2 in. by 16 ft. long with a wing nut at each end was pushed, in succession, up the outside of the stack, being fastened to the lightning cable by means of wire loops.

The first strip carried a pulley and rope. As the strips were pushed up the rod, they were fastened to each other by the wing nuts, and strip after strip was added at the bottom until the pulley and rope had reached the top. By using this rope the steeplejack pulled up another wooden strip with a large iron hook, to which was attached another pulley and a heavy line. The hook was hoisted to the top of the stack and maneuvered around until it fell over the top rim. By means of the heavy line, the steeplejack hoisted himself to the top of the stack and fastened a $\frac{3}{4}$-in. cable around the outside just below the rim. The other end of this cable was fastened to a deadman and to a $\frac{5}{8}$-in. line operating from a hoisting engine. This arrangement assured complete control of the falling stack.

The centerline of the stack was marked near the ground on the line in the required direction of fall. On the side toward which the stack was to fall, two lines of holes were drilled, the first line being about 2 ft. above the ground, and the second about 1 ft. above the first. The holes were drilled approximately 12 in. apart equally spaced on both sides of the center line. These holes, 2 in. in diameter, were drilled about two-thirds of the way through the wall of the stack and were used as blasting holes. On the opposite side of the stack a series of weakening holes were drilled, eight in all, approximately 6 ft. from the ground symmetrical about the center line, and drilled completely through the wall.

Half a stick of 60 per cent dynamite was used in each of the blasting holes. In the first blast, four holes were shot at the center, in the next two blasts eight holes were shot at one time, four on each side of the center line, and in the succeeding blasts four holes were shot at one time, two on each side of the center line. Seven blasts were required before the base of the stack had been weakened sufficiently for the structure to fall. After each blast a strain was put on the cable by the hoisting engine with 3 to 4 ft. of cable taken up at each strain. After the seventh blast, the stack stood for approximately 30 or 40 seconds and then fell. The top of the stack fell within 3 ft. of the stake which had been set as marking the spot where it should fall.

The stack was expected to fall as a rigid mass and to strike the ground in one piece. However, as shown in the accompanying illustrations, it broke in the center when reaching an angle of about 45 deg., the top section striking the ground before the base. The top section was broken into fine pieces, part of it being completely pulverized. The bottom half was fairly well broken at the midpoint, while the base was only partly broken, part of it remaining in sections large enough to require additional blasting to be removed. The reinforcing rods at the base were found to be pulled out of the concrete at the splice points near the base and were not broken. Possibly this was due to the fact that the rods were plain and not deformed or twisted.

The time required for the fall was approximately 57 minutes from the time of the first blast. Progress conformed exactly to a predetermined schedule.

The razing was done by the J. Paul Jones Company, of St. Petersburg, with the help of Salmon & Cowin, Inc., of Birmingham, which concern furnished a powder expert. The writer, as chief engineer for the Jones company at that time, supervised the work.

Asphalt Paving Conference Deals With Improvement of Secondary Roads

Producers, Technologists and Newly Formed Contractors' Association Discuss an Interesting Series of Closely Related Papers

Engineering News-Record Staff Report

LOW-COST road construction, utilizing bituminous materials of all kinds, was the subject to which practically all of the 40 papers presented at the eighth annual Asphalt Paving Conference at West Baden, Ind., Oct. 29-31, were directed. Research, finance, organization, standardization, construction methods, technique and practical experience were some of the topics covered. The following is a brief résumé of some leading papers:

Co-operative Research—Four papers dealt with research, with the participation in it of the federal government and of industry, and with the progress that has been made.

R. W. Crum, director of the advisory board on highway research, National Research Council, Washington, D. C., outlined the importance of research in finding a solution of the low-cost road problem. Study of a typical locality to determine the relative benefits is necessary, he said, to ascertain how best to raise the money and apportion it between primary and secondary roads. Surveys of traffic and of the cost of the service, as well as relative destructive effects of various kinds of traffic, are also necessary. Finally, the total cost and the annual cost of the roads to carry the expected traffic and the design of the type of road that will best carry it are at present unknown factors that must be evaluated by research.

Studies along these lines have been started, and H. S. Fairbanks, chairman of the research committee, U. S. Bureau of Public Roads, outlined what the bureau has undertaken in this respect and also what the bureau has done in attempting to standardize low-cost road improvements. These studies involve, first, broad study of a large mileage of low-cost surfaces of various types. Studies are complete with respect to surface treatments of top soil and sand clay in South Carolina and Florida, with respect to oiled clay roads in Illinois and with respect to the re-tread method in Indiana. These were made in co-operation with representatives of the Asphalt Association and several companies. The second phase is the study of efficiency of methods and equipment with the aim of suggesting improvements. The third phase is the study and analysis of bituminous materials, and the fourth phase is an intensive study of materials and methods on experimental roads.

Progress in these field studies was reported by J. T. Pauls, highway engineer of the division of tests, U. S. Bureau of Public Roads. Some of the results of this work have been reported in *Public Roads*, the bureau's magazine, and in a special bulletin of the California state Highway Department, but a full report is soon to be issued.

The part to be played by the asphalt industry in developing and perfecting bituminous products for low-cost roads and the various methods of treatment or construction of such roads were outlined by Prevost Hubbard, chemical engineer of the Asphalt Association. Special emphasis was laid on the need of weeding out unnecessary requirements in existing specifications.

The following tentative deductions were made as a result of the inspection by Mr. Hubbard of several thousand miles of low-cost road work: Success with mixed-in-place soil, gravel and similar traffic-compressed roads is obtained only with mineral aggregates of such character as will produce a serviceable road when a suitable moisture content can be continuously maintained. Waterproofing is of minor importance in arid and semi-arid localities. Where waterproofing is a major consideration, an asphalt base in place must be developed of such character as will prevent an unstable condition or an emulsion with the absorbed water, as such instability or emulsion, because of the fineness of the mineral aggregate, will produce an unsatisfactory surface. Slow-curing cut-back asphalts, with surface treatment of soft asphalt cement, appear satisfactory. As to amounts of bituminous material to use, the same laws apply as apply to hot mixtures, though a much lower minimum amount of bituminous material may be used.

All surface-treatment work should be done with the idea of waterproofing the underlying road structure. Sufficient viscosity is needed to produce a mat that will not move or be so hard as to become brittle and subject to rapid abrasion. A soft asphalt cement of between 150 and 300 penetration seems to be the most durable. On old mats a cut-back asphalt cement is most suitable. For dusty surfaces priming oil is essential.

Four basic types of asphaltic products should meet all requirements: a single grade of priming oil, three grades of slow-curing oil, two grades of rapid-curing oil and two grades of soft asphalt cement.

Methods of Low-Cost Road Construction

Ten papers related to the methods of constructing low-cost roads. They covered surface treatment, re-treading, oiling earth, blotter treatment of gravel, subgrade treatment with oil, and black base, hot-mix machine-finished and resurfaced concrete.

B. P. McWhorter, asphalt engineer of the state Highway Board, Georgia, reported the results of surface treatment of sand-clay roads in Georgia and Florida. Georgia has constructed a large mileage of chert, gravel, top soil and iron oxide silica bases at a cost of $3,000 to $10,000 per mile and used them unsurfaced for a number of years, during which time they depreciated $500 to $1,500 per mile per year. As the renewal of bases cost generally 50 per cent of the original cost, the policy now is to surface-treat all bases as soon as possible and treat all new bases within six months. Pebble soils and iron oxide silica bases give better results than any others except lime rock bases. An 8-in. base of iron oxide silica costs $3,000 to $4,000 per mile and surface treatment $2,000 to $2,200 per mile. In Florida treatments

on sand-clay two years old are still satisfactory under traffic as high as 2,500 cars per day. Re-treatments are believed to be needed after four to five years at 10c. per square yard.

Re-tread Construction—Indiana in 1929 has treated 325 miles of road with calcium chloride and 260 miles with oil. It has resurfaced 90 miles with bituminous re-tread top and 112 miles with rock asphalt or bituminous macadam, and has built 325 miles of new concrete pavement. A. H. Hinkle, superintendent of maintenance of the Indiana state Highway Commission, described in detail the present operations in re-tread construction. This consists of applying a mixture of bituminous material and crushed stone, slag or gravel made by several applications of the former to a layer of the aggregate on the old road. The mixture is kept smooth and uniform by the use of a grader or planer until the bituminous material hardens sufficiently to cause the aggregate to compact under a roller. Surface voids that cannot be closed by rolling are filled with $\frac{3}{4}$-in. aggregate as a cover just prior to the third application of bituminous material. Mr. Hinkle has been laying this type of road for eight years on the old macadam and gravel roads of the state. It costs about $2,500 per mile. Salient features pointed out by Mr. Hinkle are:

The old base is usually strengthened by 1 to 3 in. of loose aggregate with which is mixed $\frac{1}{2}$ to $\frac{3}{4}$ gal. per square yard of road oil (40 to 70 viscosity). The mixture is spread and leveled with graders, planers and long-base drags. This layer sheds water and breaks up the capillary attraction. Aggregate of $1\frac{1}{2}$- to $2\frac{1}{2}$-in. size or $\frac{3}{4}$- to $1\frac{1}{2}$-in. size is spread by graders, drags, maintainers and harrows. The harrow causes the fine material to drop to the bottom and brings a uniform coarse aggregate to the top. If the coarse aggregate is hard, it is rolled once or twice before the first application of bituminous material, which is applied in three coats, usually 0.4, 0.3 and 0.3 gal. per square yard, with perhaps a 0.2-gal. surface treatment before the following winter. Usually a cut-back asphalt of 150 to 200 viscosity or tar of 30 to 60 viscosity is used.

After each application of bituminous material the surface is rolled with a 10-ton three-wheel roller for a number of days while the bitumen is hardening. The rolling should be speeded up as the bitumen stiffens, the rate of speeding being governed by air temperature, sunshine and grade of bitumen. Application of the cover material of $\frac{3}{4}$-in. size precedes the third application of bituminous material. This cover material is spread from a truck and swept into the voids by hand or by a brush attached to a truck. Of the many pitfalls, the use of material more bituminous than is necessary for proper leveling is the most common. Too fat a mix causes the waving and rolling so characteristic of many bituminous surfaces.

In discussion, J. S. Watkins, state highway engineer of Kentucky, stated that 2,695 of 3,259 miles of road under the highway department Jan. 1, 1929, was surface-treated road. On 222 miles laid in 1928 the cost was $2,600 per mile, or $26\frac{1}{2}$c. per square yard. The few failures were caused by faulty foundations. B. E. Gray, of the West Virginia Highway Department, gave a warning that in two or three years these roads, unless properly maintained, will become rough and there will be as much complaint as from a primary road gone wrong.

Oiling—D. O. Thomas, superintendent of highways of St. Clair County, Illinois, described the Illinois practice for many years of using road oil as a dust palliative: 45,000,000 gal. is purchased annually at a cost of $2,500,000, although oiling is not considered a permanent treatment and is less certain to produce good results than many other treatments. Preparation of the road is 90 per cent of the job and oil cost and application 10 per cent, he said.

As is the case in Illinois, much money is spent in California for oiling road surfaces, but in that state an asphaltic base oil is used and frost does not militate against such treatment in winter. Chris P. Jensen, county engineer of Fresno County, California, in his paper on the subject stated that for the last eight years his county has purchased 6,200,000 gal. and the county has 3,000 miles of oiled road surfaces. The treatment is successful on soils except those containing white alkali. Roads so treated will carry 1,500 vehicles per day. Oil carrying 65 per cent bitumen is used at an average cost in place of 2.4c. per gallon. Four or five applications of $\frac{3}{4}$ gal. per square yard, each application extending over a period of two years, are usually required for satisfactory results. Cost runs to $528 per mile of 10 ft. width. Maintenance costs $50 to $200 per mile.

Bituminous surface treatments in Minnesota are used on loosely compacted sandy surfaces and heavy clay and gumbo roads, a total of 784 miles now being so treated on the primary system. The blotter method of application was described by F. C. Lang, testing engineer of the state Highway Department. After all loose material has been graded to the center, exposing on one-half of the road a smooth hard surface, $\frac{1}{3}$ to $\frac{1}{2}$ gal. is sprayed on and allowed to penetrate for 24 hours, after which a second coat of $\frac{1}{4}$ to $\frac{1}{3}$ gal. is given and at once covered (blotted) by blading back the loose material. If this is not efficacious, 350 yd. of $\frac{3}{8}$- to $\frac{1}{2}$-in. gravel or stone chips is used from windrowed stock on the opposite shoulder. The other half of the road is then similarly treated and the whole dragged several times for three to five days. Traffic is not detoured. The road will carry 700 to 1,500 vehicles per day, and the entire cost for a 24-ft. road is $1,600 to $1,800 per mile. Maintenance ranges from $400 to $1,000 per year. For clay roads oil is placed on the dry subgrade to a penetration of $\frac{3}{4}$ in. to produce supporting power.

An elaborate surface treatment of gravel and macadam roads is practiced in Oregon. R. H. Baldock, maintenance engineer of the state Highway Department, described its various phases, including the use of asphaltic oil or tar, heavy oils, road mixes, bituminous macadam and emulsified asphalt. The cost of a surface treatment of a light oil prime coat and one coat of heavy oil is 90c. per square yard and of two coats of heavy oil 94c. The light oil road-mixed type costs $1 and a multiple-lift bituminous macadam of $2\frac{1}{2}$-in. top on $6\frac{1}{2}$-in. base costs $1.18 per square yard. Traffic varying from 200 to 2,500 vehicles per day is successfully carried. Oregon, in the heavy rainfall area, has 600 miles of non-skid asphaltic concrete pavement consisting of less oil (heavy) and coarser stone at an added cost of 10c. per square yard. A home-made emulsified product has been developed by adding 10 per cent of water at air temperature immediately preceding the hot oil spray, which issues under 20-lb. pressure and 400 deg. F. temperature. Lye and jelly soap are dissolved in the water.

Subgrade Treatment—C. A. Hogentogler, senior highway engineer of the U. S. Bureau of Public Roads, discussed the highly technical physics of subgrade treatment with asphaltic oils, basing his arguments on the internal friction between particles and the cohesive resist-

ance to shear. Stability of dry clay is preserved by oil admixtures which tend to keep out moisture and to reduce capillarity. Sand or gravel admixtures increase the friction resistance. Much research based on these fundamentals is needed.

Frost-boil damage, of which little is known, was discussed by W. C. Buetow, chief engineer of the Wisconsin state Highway Department. Eruptions rarely occur in the same place, but predominate on side-hill and low-lying level roads. They may be caused by a live vein of water, or, as Mr. Hogentogler had said in his paper, by the freezing of a sheet of water below a surface crust and the attraction of water to the under-side of the sheet by capillarity. The shoulders of the road, not being traveled, do not thaw so quickly as does the center, hence an under-surface lake is formed which eventually works into a clay puddle under traffic manipulation and subsequently breaks out in geysers with ultimate disintegration of the crust. Drainage, dynamiting and cleaning out with the application of sand or gravel are practiced. Metal and wood troughs, hay, cornstacks, logs and ties are often thrown in to carry the immediate traffic.

A. H. Hinkle stated that a 2-in. layer of broken stone applied before the break-up period arrived had prevented the formation of frost boils on Indiana roads.

The use of 50,000 sq.yd. of black base on a recently filled tide-soaked riverfront drive in Charleston, S. C., was described by J. H. Dingle, city engineer. Previously a 6-in. oyster-shell road had been laid, but the subgrade oozed up through it or the shells disappeared. Flexible 4-in. base was laid on a 12-in. layer of cinders and a 1½-in. sheet-asphalt surface was applied. Since the base was placed, in 1925, some settlement, but not sufficient for discomfort in driving, is evident here and there.

Plant and Methods—Improvements in plant equipment and methods were touched upon in several papers, but were gone into in detail in the joint sessions with the Association of Asphalt Paving Technologists.

John W. Davitt, chemical engineer of the New Jersey Asphalt & Paving Company, Jersey City, N. J., indicated the newer equipment available and told of more efficient practices at plants, covering drying units, burners, fuel oil, power, mixing boxes, proportioning, storage bins and methods of adding mineral filler.

In discussion, H. C. McClure, city engineer of Flint, Mich., spoke of recent improvements in the city-owned plant from which was laid this season 370,000 yd. of 3 in. sheet asphalt. A dust collector saves 20 tons, or $100, per day of operation. A finishing machine is used to strike off both binder and top coats, doing away with the employment of eight to ten men and speeding up the amount laid to 5,000 to 6,000 yd. per day. Insulation of truck bodies has increased the permissible haul.

C. S. Pope, chief construction engineer of the California Division of Highways, described the mechanical means for spreading, raking and finishing hot asphaltic mixtures as developed in building 250 miles of road (see *Engineering News-Record*, Dec. 1, 1927, p. 869, and April 18, 1929, p. 623).

Experience of Connecticut, said W. H. Sharp, engineer of repairs of the state Highway Department, in his paper on "Resurfacing Concrete Roads With Asphaltic Wearing Courses" indicated that a minimum thickness of 1½ in. is essential on an unbroken surface. On a broken surface a 3-in. sheet-asphalt binder and top are required.

Non-Skid Surfaces—Rough-texture asphalt pavements in Europe to minimize skidding were described by F. J. Leduc, chief of the laboratory for testing materials, Montreal, Que. Stone-filled sheet asphalt using ¾-in. cubical chips after the initial compression is one device. Stamped asphalt which is rolled only in a transverse direction, and the rolling in of indentations are others.

Emulsions — Lester Kirchbraun, vice-president in charge of research of the Flintkote Company, New York, described the various types and properties of temporary and permanent emulsions, telling of the advances made in the last few years in colloid chemistry by which erratic results are now eliminated.

A paper on "Cold-Laid Asphalt Mixture," by Gen. Abson, director of the Chicago Paving Laboratory, also related to emulsified asphalts of the water-external type and to those where a volatile liquefier or a soft bitumen is used. Too little attention, he said, is given to the fundamentals involved in the use of the liquefier, particularly as to its ability to evaporate, and too much attention is given to the workability feature.

Airport Runways—All paving programs now contain papers on airport runways. In the West Baden meeting R. H. Simpson, chief engineer of the Department of Public Service, Columbus, Ohio, named the following as requirements for a standard runway pavement: reasonable cost; well-drained sub-base; base that will distribute the load; grade with no abrupt change; even, true surface; non-skid, dustless, waterproof surface; slope to drain; good visibility; capability of being easily cleaned and repaired.

Factors of service quite different from those required of vehicle pavements are required of airport runways. John A. Herlihy, chief engineer of the Transcontinental Air Transport, Inc., indicated some of these in his paper on "Oiled Runways for Airports." At sea level 1,500 ft. of a 2,500-ft. clear field must be paved, whereas at 5,000 ft. above sea level paving of 2,500 ft. of a 4,000-ft. runway is desirable. Dark oiled runways should have white borders. Hard runways for take-offs and a softer resilient surface for landings are desirable. The impact of a 40-in. 92-lb. wheel spinning 500 revolutions per minute stresses the pavement severely. Small ships with tail skids land best on softer landing strips. In arid or semi-arid areas a 2-in. mixed-in-place cake is sufficient for dust prevention, but a 4-in. layer is essential if it is to hold up under heavy rains.

Accomplishments of the Reclamation Service

Work done by the U. S. Bureau of Reclamation from its inception to June 30, 1929, as published in the *New Reclamation Era*, the official organ of the service, is summarized in the following table:

Reservoir capacity (acre-ft.)	12,881,963
Canals, ditches, and drains (miles)	16,557
Tunnels	122
Length (ft.)	175,536
Canal structures	148,462
Bridges	11,631
Length (ft.)	277,449
Culverts	14,043
Length (ft.)	535,396
Pipe (lin.ft.)	4,091,096
Flumes	4,811
Length (ft.)	853,022
Power plants	35
Power developed (hp.)	166,128
Transmission lines (miles)	2,056
Excavation (cu.yd.)	276,822,500
Irrigated farms	40,788
Population	153,663
Towns	212
Population	451,811

Economical Gas Transmission by Tapered Pipe Lines

LONG-DISTANCE transportation of nearly double the usual amount of gas for the same cost has been made possible by the development of an improved form of girth weld, the cost of which is independent of the diameter of the pipe line provided only that the weight is kept constant. The economies effected by the use of this welding method in connection with a tapered pipe line were discussed by Prof. Lloyd T. Jones, of the physics department of the University of California in a paper presented before the technical section of the Pacific Coast Gas Association.

In pipe line transmission of both liquids and gases the frictional resistance of the pipe causes the pressure to drop and gives rise to a transmission pressure gradient. Gas, however, differs from liquids in that, as the pressure drops, a loss in density occurs. Thus, if the pressure of the gas drops to half the original value, the density also falls to half, necessitating double the velocity to

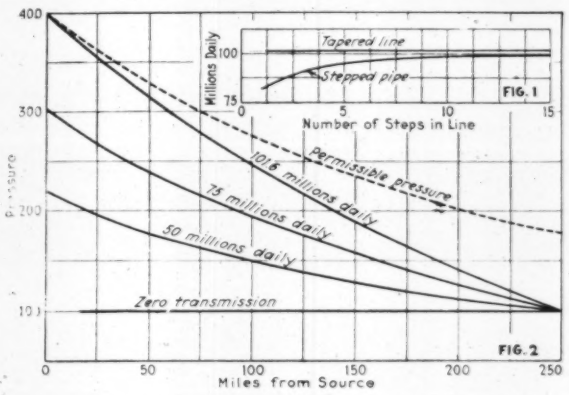


FIG. 1—EFFECT OF INCREASING NUMBER OF SECTIONS IN A STEPPED PIPE LINE. FIG. 2—GAS PRESSURES IN A 250-MILE TAPERED PIPE LINE

maintain an even flow and consequently four times the friction loss. Present practice is to recompress the gas at intervals along the line so that it may travel more slowly. It is stated by Professor Jones that the use of a tapered or stepped pipe line will greatly reduce, and possibly eliminate altogether, the necessity for recompression along the line. To prove this, he first derives general equations for gas flow through tapered and stepped pipe lines and then applies these equations to an example from actual practice, as compared with the conventional cylindrical pipe of the same total weight of metal.

All computations are based on the Weymouth formula, widely used in the calculation of cylindrical pipe lines:

$$Q_s = 15{,}385 \left[\frac{(p_1^2 - p_2^2) d^{5\frac{1}{3}}}{GTL} \right]^{\frac{1}{2}}$$

in which p_1 and p_2 indicate initial and final pressures in pounds per square inch, absolute; d = diameter, inches; G = the specific gravity of gas relative to air; T = temperature in degrees Fahrenheit (absolute); L = length in miles; Q = cubic feet transmitted daily at 60 deg. F., atmospheric pressure. By combining with the expression for gas friction in pipes, $f = 0.008d^3$, developed by Harris and cited by Weymouth in *Transactions*, Am.Soc.M.E., Vol. 34, p. 185 (1912), the following formula is obtained for a tapered pipe line:

$$Q_s = 8.0507 \times 10^6 \left[\frac{k(p_1^2 - p_2^2) d_1^{4\frac{1}{3}} d_2^{4\frac{1}{3}}}{GT(d_2^{4\frac{1}{3}} - d_1^{4\frac{1}{3}})} \right]^{\frac{1}{2}}$$

In this formula k is the ratio of increase in diameter to length, in any consistent units. For the value $k = 0$ this reduces, on evaluation of the indeterminate form, to Weymouth's equation for a cylindrical line.

Construction of a pipe line of cylindrical units in a series of steps, each slightly larger than the preceding section, will usually be more practical. The resulting line will approach the ideal tapered line in capacity as the number of steps is increased. The quantity of gas transmitted by the stepped line is given by the equation

$$Q_s = 15{,}385 \left[\frac{n(p_1^2 - p_2^2)}{GTL\left(\frac{1}{d_1^{5\frac{1}{3}}} + \frac{1}{d_2^{5\frac{1}{3}}} + \cdots + \frac{1}{d_n^{5\frac{1}{3}}}\right)} \right]$$

in which n is the number of intervals into which the line is divided. For $n = \infty$ this reduces to the formula for the tapered line, already given; for $n = 1$ (all diameters equal) it reduces to Weymouth's formula.

To bring out the advantages of the tapered construction in a numerical example, calculations are given for a hypothetical 250-mile, 20-in. pipe line designed to transport gas from an initial pressure of 400 lb. per sq.in. to a final pressure of 100 lb. per sq.in. This line, by Weymouth's formula, will transmit 62,900,000 cu.ft. daily. Tapered pipes of equal total weight will transmit as follows: 20-30 in., 101,600,000 cu.ft.; 20-40 in., 134,000,000 cu.ft. daily, increases of 61 and 110 per cent, respectively. As a tapered line of 20-40 in. diameter with the same weight as the initial 20-in. cylindrical pipe would involve final sections only $\frac{3}{16}$ in. thick, it would be more satisfactory to use the 20-30-in. tapered pipe. An increase in diameter of 10 in. in 250 miles means that each pipe length would be less than 0.001 in. larger than that preceding. It would be more convenient and suitable to increase the diameter at intervals in a series of steps, the effect of which is shown in Fig. 1. From this it will be seen that a stepped line divided into five parts of 50 miles each would approximate the fully tapered line in capacity: 96,200,000 cu.ft., as compared with 101,600,000 cu.ft. daily.

In the tapered or stepped pipe line the thickness of the steel and the pressure of the gas diminishes as the distance from the source increases. The line is so designed as to maintain constant the circumferential stress T in a pipe of radius r (inches) and steel thickness t (inches), given by the formula:

$$T = \frac{rp}{t}$$

where p is the pressure in pounds per square inch. From this formula it is apparent that the radius of the pipe may be increased at any desired ratio, the thickness of the steel decreased by the same ratio, and the circumferential stress will remain constant provided the pressure is decreased by the square of that ratio. The initial section of the 20-30-in. tapered steel pipe, $\frac{3}{8}$ in. thick, under a pressure of 400 lb. would have a stress of 10,267 lb. per sq.in. At the terminus the 30x$\frac{1}{4}$-in. steel at a pressure of 186 lb. per sq.in. would have the same stress, 10,267 lb. per sq.in. (In the case of the stepped pipe there will be slight variations.) If the final pressure is reduced to 100 lb. per sq.in., stresses all along the line will be lower. Conditions obtaining under these circumstances are shown in Fig. 2.

Another advantage of the tapered pipe is that it has a greater storage capacity than the corresponding cylindrical line, the initial diameters being identical.

From Job and Office

Hints That Cut Cost and Time for the Contractor and Engineer

Hinged Jib on Crane Boom Extension Handy for Steel Erection

A 17-FT. hinged jib mounted on the end of a 20-ft. extension to a regular 45-ft. crane boom made possible the erection of all the steel for a six-story narrow building in the Bronx, New York City, from the street without the use of any other equipment. Both the rigid and the movable boom extensions were developed by Thomas Foley, a hoisting engineer for Kues Bros., Bronx, N. Y., subcontractors on steel erection for the building. A view of the crane with a load is shown in Fig. 1.

The rigid 20-ft. extension consisted of two 10-in. channels 30 ft. long crosstied with angles and riveted to the side plates at the end of a 45-ft. lattice boom on a 15-ton gasoline crawler crane. This extension was hog-rodded from an angle-iron horseback to the bottom of the main boom to provide additional stability. On the end of the rigid extension were riveted two $\frac{5}{8}$-in. plates, making a boom seat with $1\frac{1}{2}$-in. pin. This pin held the movable jib, built up of two 8-in. channels with $\frac{1}{4}$-in. plates as tie bracing. A horse was built up of heavy angles on the lower end of the jib. Cables from the top of the horse to turnbuckles at the base of the main boom held the jib in position. By manipulating the turnbuckles, the position of the jib with respect to the rigid extension could be changed from straight to 90 deg. This feature gave the crane additional radius as well as altitude, permitting the handling of steel one bay back of the street face of the building. The turnbuckles are shown in Fig. 2.

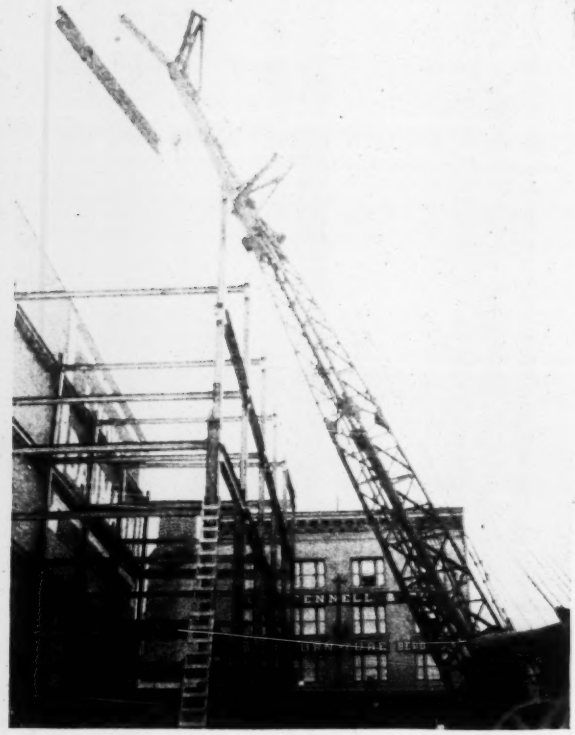


FIG. 1—17-FT. HINGED JIB ON CRANE BOOM EXTENSION

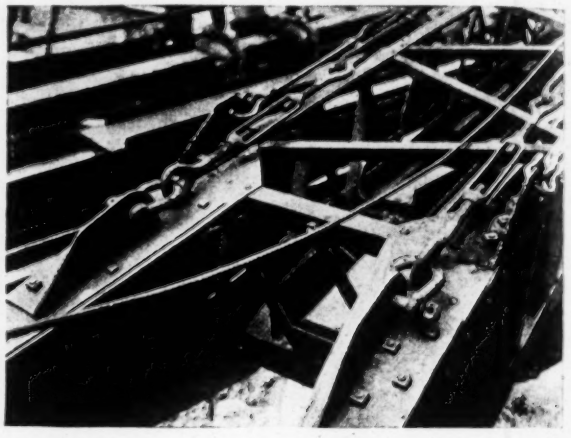


FIG. 2—TURNBUCKLES FOR CONTROLLING HINGED JIB

The building on which the crane was used is a six-story store and warehouse being built at 152d St. and Third Ave. by the Commonwealth Engineering Corporation for D. Bauman & Company.

Compiling and Preserving Detailed City Engineering Data

By R. V. Newcomb

City Engineer, Vermillion, S. D.

TO AVOID the accumulation of tracings, blueprints, maps and miscellaneous data which usually clutter up a city engineer's office a plan has been developed in Vermillion for recording detailed information on sewers, water mains, light conduits, gas lines, street grades, pavements and similar data on a series of loose-leaf maps. These are made on tracing cloth $10\frac{1}{2}$x12 in., one sheet to a block. The transverse street scale is 1 in. = 30 ft., while the longitudinal street scale is 1 in. = 60 ft. To aid in separating the data, a color system is used as follows: black for lot outlines and dimensions, sidewalks and pavement; brown for all sewer data; blue for water main service connections and hydrants; red for electrical conductors; green for actual measurements and for plusses to various service connections.

This system is easily kept up to date. When radical changes are made in any area, such as replatting or cutting through new streets, it is not necessary to erase or to draw conflicting lines. The old map is simply marked "Superseded by revision dated ——" and a revised map of the block made and placed in a position preceding the old. Thus a complete record and history of the engineering development of the city is available without the necessity of making a new map every few years. For an index, tabs are placed marking the various additions, and the maps are then run in order of block numbers. For unplatted sections, the maps are collected together and arranged according to the alphabetical order of the

streets upon which they border. The use of tracing paper makes it possible to make blueprint copies for the use of other city officials, departments, and public utility companies at any time.

Supplementing the loose-leaf detail maps, small-scale individual maps of the entire city are used to show the sewers, water system and other utilities.

Floating Timber Frame Used as Guide for Dumping Riprap From Scows

RIPRAPPING a submerged slope at the toe of a concrete bulkhead wall, recently built at Portland, Ore., as a part of the Front St. intercepting sewer system, involved dumping 29,000 cu.yd. of rock over an area 2,300 ft. long by 50 to 200 ft. wide. In order to secure the specified minimum thickness of 2 ft. uniformly without waste, a floating timber guide frame was used through which skiploads of rock were dumped.

The timber frame consisted of 50-ft. logs bolted together to form a square, with intermediate logs dividing it into sixteen bays 12x12 ft. in size. Stiffness was secured by a horizontal timber truss attached to one side.

This floating frame was located by transit and anchored, and the riprap, which consisted of quarry-run rock delivered in scows, was dumped through the openings in the frame. Three skiploads of 3½ cu.yd. were deposited through each 12x12-ft. opening. Inspection by divers showed this quantity of rock and method of application gave a uniform distribution of riprap in the required thickness, according to F. T. Fowler, bridge engineer, Portland, Ore. When the area occupied by the frame had been covered, it was moved to a new position and anchored and the process was repeated. The water depth varied from 30 to 85 ft. over the area protected and the river bottom slope was approximately 3 to 1.

Concrete Plug Drilled and Blasted Under 180-Ft. Head of Water

ONE MAN working in a 6-ft. penstock successfully carried out the hazardous task of removing a concrete plug, 6 ft. thick, that was supporting a 180-ft. head of water, in order to open the penstock for power use. The plug had been poured during the construction of the Don Pedro dam to blank off this additional penstock line to the power house, and the subsequent need for power presented the difficult problem of removing the plug without drawing down the reservoir because of the value of the stored water. The plan finally used consisted of honeycombing the concrete with drill holes so that a large number of small charges would break up the plug without disturbing the surrounding concrete of the dam. Use of a diver working from the outside was not feasible because of the head of water that could not be reduced, and after various attempts to place a bulkhead on the upstream face to reduce the pressure had proved unsuccessful, the only alternative left was to remove the plug by working from the penstock toward the upstream face that was sustaining a 180-ft. head. The scheme used was devised and executed personally by T. E. Connolly, of San Francisco, the contractor for the work, when difficulty was experienced in securing help to do the job.

The Don Pedro dam, on the Tuolumne River in central California, is a part of the storage development of the Turlock Irrigation District and was built in 1921-22, at which time two penstocks for future power use were left through the dam, in addition to those put into service, and were plugged by pouring the upstream 6-ft. section solid with the dam.

The decision to develop this additional power without loss of stored water presented a difficult problem to the contractor in the removal of the plugs. The first plan tried was to put a seal around the plug by means of a bulkhead lowered from the crest of the dam. This proved unsuccessful because of an obstruction on the upstream face that would not permit the bulkhead to seat, and an attempt at putting leverage on the diaphragm did not help. In an effort to seal this bulkhead, three 4-in. pipes equipped with valves were grouted into holes drilled at the edge of the plug and were then drilled

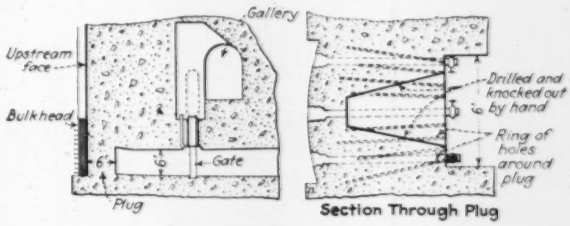


DIAGRAM SHOWING LOCATION OF PLUG AND ARRANGEMENT OF DRILL HOLES

through to pierce the concrete so that, by opening the valve, suction could be used to pull sawdust and straw into the space around the bulkhead. This also proved unsuccessful in lessening the pressure as indicated by the flow from the three pipes.

Any plan to remove the plug by shooting it out from the solid would have required a charge of powder that might endanger the surrounding concrete at the base of the high dam. Difficulty in reducing the pressure further complicated any scheme to remove it in sections. For this reason a wedge-shaped central section, shown in the accompanying sketch, extending to within about 18 in. of the upstream face was drilled and knocked out by hand. This permitted the remaining shell to be shattered by small charges. Next a ring of holes was drilled around the edge of the plug directed away from the center line as a safety precaution so that in case the plug should break loose as this drilling neared completion it would wedge itself in the hole, preventing a sudden flood. The 1¼-in. drill holes formed an almost continuous ring and separated the plug from the rest of the dam with a plane of weakness, tending to keep the force of shooting from entering the mass of concrete. They were drilled as close to the face of the dam as possible, governed by the calculated thickness and the observed behavior of the drills. Finally, additional holes were drilled to take out the edge of the wedge and to finish cutting the plug up into small sections. All this work had to be done, using, as a means of ingress, the difficult route through the inspection gallery, into the slide gate operating chamber 15 ft. back from the face of the dam and thence down into the penstock to the face, the slide gate previously having been removed. A helper working in the slide gate chamber kept Mr. Connolly supplied with drill steel.

After the drilling was completed the slide gate was reassembled and opened wide. The penstock was entered from the lower end and the holes were loaded.

The charges consisted of from two to six sticks of 50 per cent dynamite in every fourth hole, using several sets of delay exploders timed to fire from the center outward. This reduced as much as possible the risk from a concentration of explosions. The wires were run up through the bypass to the slide gate operating chamber and the shot was fired from this point; the butterfly valve at the lower end of the penstock previously had been set slightly ajar.

Standing on top of the slide gate 15 ft. from the plug, the shots were indicated by a slight thud. The bulkhead had been left in place to check the rush of water and after the shots the penstock filled rapidly. The slide gate was then closed and the butterfly valve was opened; inspection showed that the plug had been shattered into 6-in. pieces. Sections of all three of the pipes were found, proving that the plug had been completely removed. The same method was used to break out the second similar plug to open up another penstock.

Concrete Culvert Moved to New Location by Stump Puller and Rollers

By John Calhoun
Bridge Foreman, State Highway Department, Bradford, Ark.

In relocating a state highway near Heber Springs, Ark., a 3-ft. box culvert 32 ft. long was removed a distance of 41 ft. by a stump puller and rollers at a total cost of $149.40. A new culvert for this location would have cost $450 or $500. The box, estimated to contain 14 cu.yd. of concrete, was moved upstream directly along its center line with no sidewise displacement. An apron and wing walls extended 5 ft. from both ends of the box, the wings being parallel on the downstream end and flared out on the upstream end, with a cross-toe wall at the extreme end. The downstream toe was cut loose from the structure by light charges of dynamite.

The first operation in moving the box was to jack up the upstream end with three screwjacks placed as shown in the accompanying illustration, one under each side, well forward, and one under the front center. Pry poles heavily weighted were used to help raise the box. Peeled pine rollers 6 to 8 in. in diameter were slipped under the forward third of the structure, resting on longitudinal planking, and the load was transferred from the jacks to these rollers. The rolls were spaced about 5 ft. centers. Trenches were dug across the route, spaced to allow the roller to drop before passing the [illegible] point of the box, thus keeping the front end high at all times.

Pulling was done by an ordinary stump puller and a team of mules. The cable was attached to the [illegible] through a hole in the floor by means of a hook and cross-bar. The total moving time was less than 30 minutes, a rate of 400 ft. per hour being maintained while the structure was in motion.

The detailed costs of moving the culvert follows:

Assembling equipment and labor	$16.25
Rent of equipment	5.40
Detour	13.50
Lifting culvert	28.50
Dynamiting toe wall	4.75
Moving culvert	13.65
Excavation	59.85
Backfilling	7.50
Total	$149.50

The assemblage, excavation and detour costs would have been practically the same had a new culvert been built.

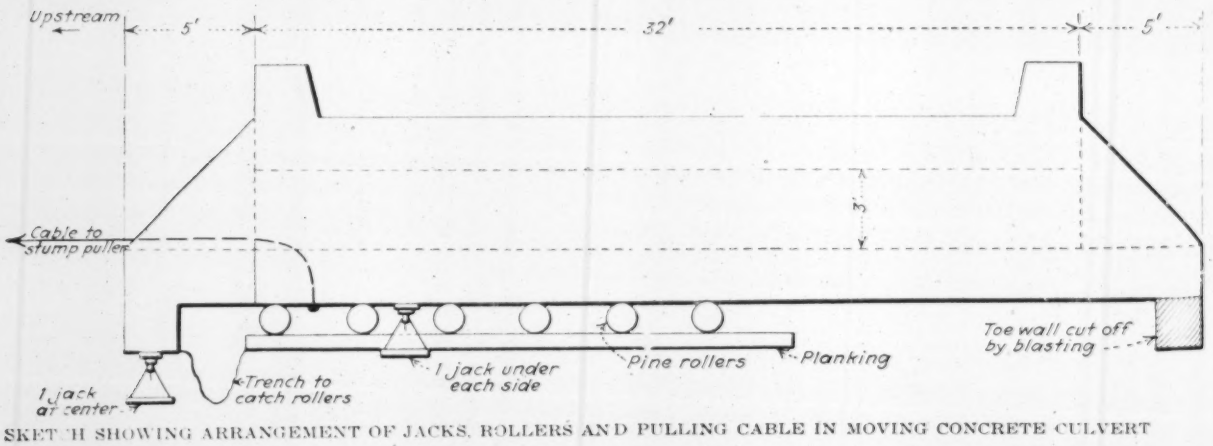
The success of this venture indicates that larger culverts could be moved with equal facility by the use of heavier equipment, especially the jacks, preferably hydraulic.

This work was done by the author under the supervision of Cone Murphy, district engineer of the fifth district, Arkansas Highway Department, and Mr. Roby, assistant engineer of the district.

Divers Place Pipe in 60 Ft. of Water on Seattle Sewer Job

A 700-FT. length of cast-iron outfall sewer pipe 4 ft. in diameter recently was placed in the east waterway at Seattle, in water depths ranging up to 60 ft. Because of soft bottom, the pipe had to be supported on piles which were driven in a trench. The work carried on by divers and the method of placement, described in the following, was worked out to enable divers to get the pipe sections fastened quickly, securely and accurately on grade.

The first operation was excavating a trench 12 to 18 ft. deep in the harbor bottom in which piles were driven to support the pipe line. Despite the fact that the excavation was made extra wide and the pipe-laying operations, followed immediately by backfilling, were kept as close as possible to the dredging, the sides of the excavation sloughed badly, necessitating redigging in some cases. At other points the piles were undercut by



SKETCH SHOWING ARRANGEMENT OF JACKS, ROLLERS AND PULLING CABLE IN MOVING CONCRETE CULVERT

tide-wash before the pipe could be placed, necessitating filling in with gravel.

The piles were driven in pairs 12 ft. apart along the center line and a pair of collars or sub-caps consisting of 6x12 timbers was fastened across each pair of piles at the proper level to serve as a support for the pipe. This design put a collar support under each 12-ft. length of pipe. The collars were bored on the wharf before being sent down to the diver and were attached to a rod marker that would insure the right depth. The marker projected above the water surface so that a level instrument could set it at exact grade. On signal the diver spiked the sub-cap to hold it temporarily and then put on clamps to fasten it in place pending final bolting. The hole in the sub-cap served as a guide for the auger with which the diver bored through the pile. For this work an air-driven drilling machine well covered with heavy cup grease to keep out the water was used and was found entirely satisfactory. After the hole had been bored the diver completed the fastening by a 1-in. bolt passed through pile and collars.

The pipe was lowered into the water in 36-ft. lengths consisting of three 12-ft. sections that had been fastened together on shore. The sections weighed a little more than 4 tons each, or about 13 tons per 36-ft. length. Joints between lengths were made by divers.

As an aid to the divers a white line was marked around the spigot end of each joint to be laid under water. This facilitated making sure that insertion to the proper depth was secured in the dim light by which the divers worked. All subaqueous joints were calked by divers with lead wool. After the pipe was in place piles were cut off even with the top of the pipe and the trench was backfilled with gravel and sand.

The work was done under contract by the General Construction Company, of Seattle.

Job and Office Note

CANVAS TRAFFIC GUIDES have been successfully used in Washington, D. C., and have proved more satisfactory and more economical than paint. The cost of the material ranges from 6c. to 7c. per running foot, and the labor cost of installation is small compared with the cost of painting. The heavy canvas is impregnated with paint and coated on one side with an adhesive substance. Approximately half an hour is required for the application to dry, but traffic is permitted to roll over the tape as soon as it is laid.

Letters to the Editor

Beauty as an Ideal

Sir—Permit me to supplement your editorial "Beauty as an Ideal," in your Oct. 3 issue and to record my version of the question raised in the first sentence, "Beauty in engineering work—what is it?"

A prevailing truth was expressed when you stated that engineering structures have often been called severely plain and sometimes ugly. The recent competition for architectural designs for steel bridges open to students and sponsored by the A.I.S.C. obviously takes cognizance of this, and it also recognizes the fact that the creation of a harmonious structure is a practical and desirable process. Too often the engineer conceived his bridges to fulfill the immediate purpose for which they were to exist, and out of the bedlam of slide rules, detached studies, specifications and contracts his structure arose, purely utilitarian and nothing more. He has been too engrossed in the structural side of the problem; he has been actuated by a desire to keep down costs; he did not interest himself in esthetics, or he has not been trained in the history and principles of the art of proportion and harmony. Often he believes that enhancement is necessary only when work involving great sums is carried out. As a result thousands of bridges possess function but are barren esthetically. It is also akin to saying that the only field for the display of architectural dignity is in the larger and more important public and private buildings. This is a view erroneous in the extreme, for one poorly designed structure may ruin an entire landscape.

Then again engineers have possessed the notion that architecture by the yard may be added to a bridge after the major problem of design and drawings is completed. This of course is a myth, for a structure may be plastered with decoration and still be ugly.

One thousand years represented the span of the most perfect civilization the world has ever known. The Greeks created and developed art and taught the world to appreciate beauty. The succeeding Roman civilization endured for 600 years and the charm of the bridges across the Tiber have been perpetuated in stone for the appreciation of this generation.

The lack of proper architectural treatment in bridges is only a lapse. Bridges as an art have always existed; as a science they are recent.

The day of the pile trestle, the old steel truss, the weak and sagging timber bridges, the structure built for purely utilitarian reasons, is largely a thing of the past. Architectural dignity more than anything else will distinguish the bridges of today from those of the recent past. And architectural dignity is not applied ornament. Beauty—what is it? It is often the inherent charm in the graceful lines of a concrete arch. It is more often the ability to mold the essential structural forms in such a way as to perform the main purposes for which the structure exists plus the creation of a pleasant reaction to the eye. It means that the engineer adequately trained, or the architect and engineer in collaboration, must control the shape and placement of structural features of a bridge as they are evolved, thereby insuring for the finished structure a continuity of design not otherwise attainable.

The mistakes of man in many professions are eased with the advent of time, but the mistakes of the bridge designer live in concrete and steel to haunt him year after year, a glaring symbol of his inadequacy. Most people will recognize ugliness at a glance, but too late to correct it. Those trained in the principles of art can detect the potential ugliness in drawings, and until the engineer receives art training my conclusion is that the interests of all may be better served by collaboration between the engineer and architect.

Trenton, N. J., Oct. 7, 1929. ARTHUR J. LICHTENBERG, Bridge Designer.

* * *

Sir—I have just read your editorial entitled "Beauty as an Ideal" on p. 518 of your issue for Oct. 3, and I want to take this opportunity of congratulating you upon what appears to me to be a clear and true statement of the case regarding the present situation in what may be called "engineering architecture" for the lack of a better term.

The ideas and opinion expressed in your editorial are such that I can personally indorse 100 per cent, and I am convinced that the wide publication of your editorial will very materially assist in bringing about a better realization, among engineers especially, of the subject.

F. H. FRANKLAND,
Manager of Technical Service,
American Institute of Steel Construction, Inc.

New York, N. Y.,
Oct. 5, 1929.

Discussion and Rebuttal on the Hackensack Bascule Failure

Following the failure of the east leaf of the double-leaf Hackensack bascule bridge when it dropped into the river on Dec. 15, 1928, the following articles having appeared in *Engineering News-Record:*

News account of failure, Dec. 20, 1928, p. 931.

Preliminary report of investigating board, Jan. 10, 1929, p. 74.

Designers differ with board on cause of failure, Feb. 14, 1929, p. 268.

Final report of investigating board, June 6, 1929, p. 916.

Analysis of dynamic stresses, by C. E. Paine, Aug. 29, 1929, p. 340.

Investigation board discussion of counterweight tower action, Aug. 29, 1929, p. 343.

The accompanying letters, including a reply from the investigating board, carry the discussion of this most remarkable failure further.—EDITOR.

Calculated Stresses in Counterweight Tower

Sir—The analysis of the forces produced by deceleration of the counterweight in the Hackensack River bridge, as given by C. E. Paine in your issue of Aug. 29, p. 340, solves an unusually interesting and clear-cut problem in mechanics. Mr. Paine's expression for y_{max} may be somewhat simplified as follows:

$$y_{max} = V_o b \sqrt{2}\, \frac{1 - \cos \sqrt{\frac{1}{b}}\, t_o}{t_o} = V_o \sqrt{b}\, \frac{\sin \frac{1}{2}\sqrt{\frac{1}{b}}\, t_o}{\frac{1}{2}\sqrt{\frac{1}{b}}\, t_o}$$

$$= V_o \sqrt{b} \left[\frac{\text{chord}}{\text{arc}} \text{ of angle } \sqrt{\frac{1}{b}}\, t_o \right] = K V_o \sqrt{b}$$

where K is a function of t_o. This shows by inspection why this coefficient is nearly constant up to $\sqrt{\frac{1}{b}}\, t_o = \pi$ and also that the absolute maximum value is unity for $t_o = 0$.

Another interesting relation may be deduced from the equations given by Mr. Paine:

$$F_{max} = M \frac{y_{max}}{b} = M \frac{K V_o \sqrt{b}}{b}$$

$$F_{max} \propto \frac{1}{\sqrt{b}}$$

If t_1 is the period of oscillation

$$t_1 = 2\pi \sqrt{b}$$

$$\frac{1}{\sqrt{b}} = \frac{2\pi}{t_1}$$

Hence F_{max} varies inversely as the period of oscillation. If the period of oscillation can be accurately determined either by computation or by measurement in the field, the absolute maximum value of the decelerating force acting on the counterweight can be directly computed from elementary principles of mechanics.

Mr. Paine computes 3.12 seconds for the period of oscillation, but the investigating board gives a measured value of 4.13 seconds. If we accept the latter value, the decelerating force in Mr. Paine's computations should be divided by $\frac{4.13}{3.12} = 1.32$. The board has erred in multiplying by 1.32, for it is evident, without reference to Mr. Paine's analysis, that with the more flexible tower—longer period of oscillation—a smaller decelerating force is needed to absorb the kinetic energy of the counterweight, since the force acts through a greater distance.

The board also errs in assuming that the ratio of decelerating force to resulting stress is the same for the two periods of oscillation. The tower stress is a result both of the decelerating force and also of the eccentricity of the counterweight which accompanies deflection of the tower. If the period of oscillation is longer, this shows that the tower is more flexible; the decelerating force then is smaller, as is also the stress due to this decelerating force, but the stress due to the eccentricity of the counterweight is relatively greater. If the period is 32 per cent greater than that computed by Mr. Paine, the stress due to counterweight eccentricity for a given decelerating force will be more than twice as great as before. For small changes in the period it is found that the total stress resulting from deceleration is nearly constant.

For tower stress resulting from deceleration of the counterweight, the following different values are reported:

	Lb. Per Sq. In.
Final report of the investigating board:	
Bridge stopped by motor brakes	27,000
Bridge stopped by emergency brakes	37,600
Bridge stopped by motor brakes and emergency brakes	64,600
Bridge stopped by motor brakes, and emergency brakes and friction	69,600
C. E. Paine, for a period of oscillation computed as 3.12 seconds:	
Bridge stopped instantly	22,200
Bridge stopped as in any of above cases, from his curve, about	21,000
Board—revised value for a period of oscillation of 4.13 seconds (measured):	
(Based on its assumption that the ratio of decelerating force to tower stress is the same for the two periods of oscillation) $22{,}200 \times 1.32$	29,300
Corrected value which board should have deduced for a period of 4.13 seconds, from the erroneous assumption just stated $22{,}200 \times \frac{1}{1.32}$	16,800
Value for a period of oscillation of 4.13 seconds with allowance for change in eccentricity of counterweight:	
Bridge stopped instantly	21,500
Bridge stopped by any combination of brakes, about	20,000

This seems an unreasonably wide variation of solutions for a definite problem in mechanics. It is hoped that the board will explain the method of computing the values for tower stresses due to deceleration as given in its final report.

Tower stiffness affects also the stresses in the links and link pins resulting from deceleration of the counterweight—the stiffer the tower the greater are these stresses. If for the stresses from this source, with the tower as designed, the values given by the board in its final report could be deduced, it is pertinent to ask what stresses exist in the links and link pins with the tower stiffened as recommended by it.

Urbana, Ill., Sept. 14, 1929. HARDY CROSS, Professor of Structural Engineering, University of Illinois.

* * *

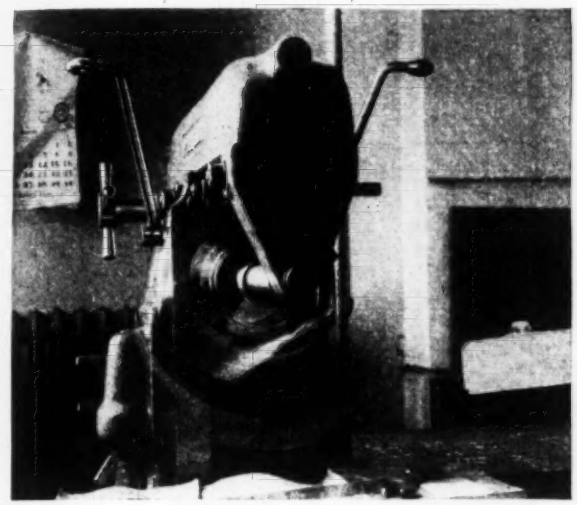
Behavior of a Pendulum Suspended From a Rotating Shaft

Sir—In *Engineering News-Record*, Sept. 12, 1929, p. 426, attention is called to the fact that the trunnion friction of the Hackensack bascule could not be depended upon to damp the oscillations of the counterweight. By keeping in mind the rule that dry-surface friction is independent of speed, it is very easy to understand the behavior of the model shown. The friction exerts a constant torque on the pendulum, and the angular momentum imparted during the forward stroke is exactly neutralized by the angular momentum taken away during the back stroke. With a lubricated bearing, however, the case would be different. Since, for low speeds, the friction decreases rapidly with speed, and the relative speed being greater during the back stroke than during the forward stroke, the angular momentum imparted during the forward stroke would be more than that taken away during the back stroke. Consequently, the amplitude of the vibration would increase as long as the angular speed of the shaft is greater than the maximum angular speed of the pendulum.

To test this conclusion, a model similar to the one made by Professor Beggs was set up with a motor and worm gear drive capable of speeds from 4 to 40 revolutions per minute. The model had a ½-in. steel shaft with a well-fitted (but not tight) brass bearing from which hung a pendulum weighing 11 lb. and having a period giving 70 vibrations per minute. At all speeds up to 40 revolutions per minute, without an initial start except that given by friction on starting the apparatus, the vibrations built up until the maximum angular speed of the pendulum slightly exceeded that of the shaft, or, in other words, until the minimum velocity of the pendulum relative to the shaft became negative. Lubrication was freely supplied with an oil can through a hole in the top of the pendulum bearing.

The expected amplitude of the vibration can be worked out by a very simple rule: Mark on the pendulum the point at which the amplitude is to be measured. Then the expected amplitude times the vibrations per minute equals twice the distance from the marked point to the center of the shaft times the number of revolutions per minute of the shaft.

When the speed of the shaft is large and the amplitude of the pendulum is small, there is a smaller relative dif-



EXPERIMENTAL SET-UP OF A HEAVY PENDULUM SWINGING FROM SPINDLE OF A MILLING MACHINE

ference between the maximum and minimum relative speeds. This accounts for the fact that the amplitude builds up more quickly with low speeds and that it builds up with increasing rapidity as it approaches the maximum. At speeds above 40 revolutions per minute the vibrations would not start automatically but built up readily if given a start. By changing the drive to get higher speeds and by gradually speeding up the motor to keep the speed of the shaft greater than the maximum speed of the pendulum, the amplitude was built up until the pendulum went entirely over and took up the speed of the shaft.

Without lubrication, the amplitude does not build up, but remains practically constant for any start up to the maximum angular speed of the shaft. With a ball bearing (clean, dry, and new), there is a very slight tendency to build up.

STRENGTHENED COUNTERWEIGHT TOWER, HACKENSACK BRIDGE

A-frame structure is new. Original counterweight tower is laced vertical member in center of A-frame.

With cup grease lubrication, the small model would not build up, and the experiment was tried on a larger scale.

A pendulum weighing 101 lb. was hung from the spindle of a milling machine shown in the accompanying illustration. The collar bearing was 1¼ in. in diameter, and of hard steel, ground to a running fit on steel spindle ¼ in. wide, giving a bearing pressure of 322 lb. per sq.in. of projected area. With good grease lubrication, the spindle was started at 20 revolutions per minute. No initial push was given the pendulum, and the following readings were taken:

Time From Start, Seconds	Amplitude of Vibration 10 In. From Center of Shaft, In.	Time From Start, Seconds	Amplitude of Vibration 10 In. From Center of Shaft, In.
1	¼		
20	½		
30	1	80	4
50	2	90	5
65	3		

The following table shows the maximum amplitudes resulting from three speeds of the spindle:

Shaft Speed, R.P.M.	Pendulum Speed, V.P.M.	Amplitude at 10 In. Expected, In.	Amplitude at 10 In. Actual, In.
20	64	6¼	6¼
27	64	8⅝	9¼
35	63	11	14

Oil lubrication gave about the same results.

At any of these speeds the vibrations would start automatically when the spindle was started, the rate of building up being slower with the high speeds, as in the small model.

The whole experiment strikingly illustrates the observations of Goodman on friction of lubricated surfaces, in his "Mechanics Applied to Engineering," 4th Ed., pp. 247-248.

Columbus, Ohio,
Sept. 21, 1929.

P. W. Ott,
Department of Mechanics,
Ohio State University.

* * *

Reply of Investigating Board

Sir—The board appreciates the discussion contributed by Professor Ott, confirming and strengthening previous experimental and analytical conclusions as to the persistence and augmentation of oscillations in the elastic counterweight towers of the Hackensack River bascule bridge.

Referring to the discussion by Professor Cross, the board does not care to be drawn into any secondary controversies involving juggling of formulas or mathematical quibbling. The evaluation of maximum stresses in the counterweight towers of the Hackensack River bridge is more than a "clear-cut problem in mechanics," inasmuch as even the most advanced mathematical solution is valueless if the underlying assumptions are incorrect or incomplete. The board has already shown that Mr. Paine's analysis was incorrect or incomplete in his erroneous assumption that oscillations would be damped by friction and that their effect may therefore be ignored. This important point emphasized by the board has since been independently confirmed by the investigations of Professor Ott, who shows that, with lubricated bearings, the oscillations are subject to serious augmentation. There are also other factors that would make the stresses much higher than those submitted by Mr. Paine. In fact, under the action of certain conditions that should be considered in a complete investigation of the safety of the design, the board is prepared to show resultant stresses greater than any that have thus far been reported. In view of these facts, the board reiterates its conclusion that the structure as designed was subject to excessive and dangerous stresses.

The accompanying illustration shows the counterweight tower structure as rebuilt following the failure. The vertical member, consisting of a pair of laced 12-in. channels, in the middle of the A-frame, corresponds to the original counterweight tower. The board is pleased to report that actual stress measurements in the tower and the top link since the structure was stiffened show unit stresses due to full braking deceleration to be materially lower than those observed before stiffening the tower.

Board of Engineers,
O. E. Hovey,
George E. Beggs,
D. B. Steinman.

News of the Week

Prequalification Held Illegal in Philadelphia

Work on city improvements in Philadelphia, Pa., amounting to nearly $5,000,000 is being delayed by a decision of the Court of Common Pleas that prequalification of bidders is illegal. Although the decision was rendered in the case of a contract for the Ridge Ave. subway, awarded to the Keystone State Corporation under the prequalification provision, the decision has affected a number of other contracts awarded and pending. Appeal to a higher court is considered probable.

Contract Awarded for Canadian Grain Elevator

Construction of the 5,500,000-bu. elevator which will complete the dominion government's initial plans for creation of a Great Lakes terminal grain port at Prescott, Ont., will be carried out by the Atlas Construction Company, Ltd., of Montreal. This company's bid of $3,093,000 was the lowest of several submitted by prominent Canadian contracting firms. According to the terms of the contract, construction must be completed by Aug. 1, 1930.

Bonus Offered to Speed Work on Wanaque Aqueduct

In order to hasten completion of the Wanaque aqueduct in northern New Jersey, the North Jersey District Water Supply Commission has agreed to pay the T. A. Gillespie Company, contractor on the job, a bonus of $2,000 a day for completion before the contract date, July 24, 1930. The bonus, however, is limited to a period between 30 and 120 days, and no premium will be paid unless the work of laying the twin steel pipe is completed at least 30 days before the date specified in the original contract, which was awarded Jan. 24, 1929. Since this date the aqueduct has been more than two-thirds completed, although only about half the time allowed has elapsed. Rapid progress has been due to favorable weather conditions.

Chicago Bond Issues Defeated

Of seven bond issues and one increased-taxation proposal aggregating $50,000,000, voters of Cook County, the city of Chicago and two Chicago park districts on Nov. 5 approved only a bond issue of $3,000,000, this being for the enlargement and improvement of Lincoln Park along the waterfront under the Lincoln Park District. The largest project voted on was for a $20,000,000 "elevated pleasure highway or boulevard" from the business district west to the city limits, to be built by the West Park District. Other defeated projects were $3,320,500 by Cook County as its share of a program for 38 highway grade separations and eight combined highway and railway grade separations, it being assumed but not known that the state would provide $4,300,500 to the same end; three bond issues totaling $18,876,000 by the city to pay judgments already obtained by property holders for damages due to the widening of Ashland Ave., La Salle St. and Western Ave.; $2,500,000 for the Cook County forest reserve, these bonds to provide shelter and comfort houses, water supply and other necessities and conveniences, and to run only five years; and a $2,500,000 tax increase in Cook County for one year to pay the cost of the reassessment of property which is now in progress. The Chicago Bureau of Public Efficiency, in a pre-election pamphlet outlining these eight proposals, advised voting "yes" on all but the elevated highway bonds, opposing these because, among other reasons, it felt the improvement should be carried out either by the county or by the city rather than by the park district. Three proposals for changes in number of municipal court judges, aldermanic elections and city treasurer's duties were also defeated.

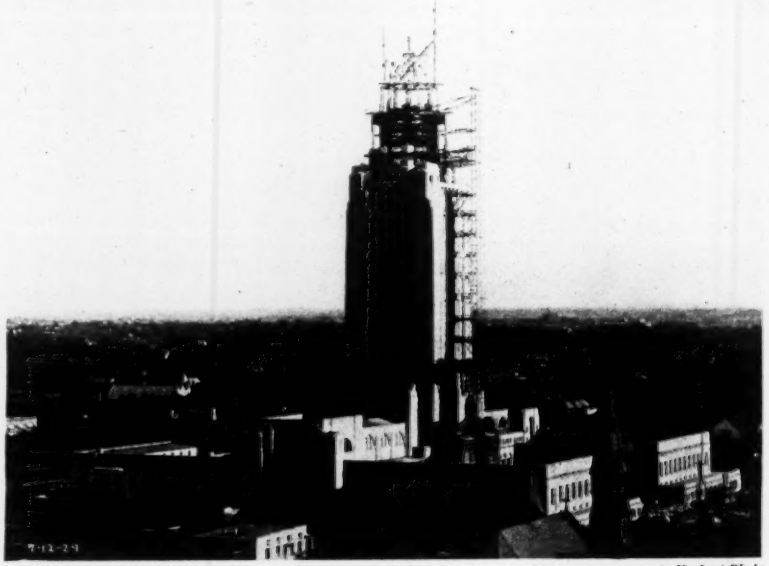
San Francisco Votes Both For and Against Garbage Incineration

Voters of San Francisco on Oct. 6 approved incineration of garbage by a good majority, but at the same time gave much larger majorities against either a municipally owned system of garbage disposal (kind not specified) or the city contracting for the private construction and operation of an incinerator. They also voted heavily against a municipally owned and operated system of collection.

These were "policy" votes, on referendum directed by the board of supervisors (city council) and have no legally binding effect. An outline of the various proposals was given in *Engineering News-Record*, Sept. 26, 1929, where it was also stated that the supervisors had voted 13 to 5 against a referendum on the fill-and-cover method of disposal. Later a mandatory fill-and-cover ordinance, applying to refuse instead of garbage only, was placed on the ballots by means of an initiative petition, but it was defeated Nov. 6 by 51,000 votes against to only 27,000 for.

This ordinance would have required the supervisors to buy land and use it for refuse disposal by the fill-and-cover

NEBRASKA'S CAPITOL TOWER NEAR COMPLETION



Herbert Photo

Eight years have elapsed since construction was started on the new Nebraska State Capitol building at Lincoln, due to the fact that it is being financed from current funds without bond issues. The structure is a radical departure from usual design, consisting of a hollow square one story in height inclosing a cross-shaped structure containing legislative halls and courtrooms. Between these two are four open courts which furnish light and air to the interior. At the center is a 400-ft. tower. The entire structure, which cost approximately $11,000,000, covers nearly 4½ acres of ground. Completion is expected in January.

method for two years, when a vote on buying more land for the purpose would have been taken, the method named to be abandoned if the second vote was adverse. It was urged against this ordinance that it was impossible to buy the first part of the disposal area within the stipulated period of two months and that the second election required made the whole scheme a gamble. The vote on the five proposals, according to the first count by the election commissioners, was:

	Majority	Yes	No	Total
Shall garbage be disposed of by incineration	+9,105	42,284	33,179	75,463
Shall the city establish a municipally owned and operated system for garbage collection	—43,372	12,790	56,162	68,162
Shall the city establish a municipally owned system for garbage disposal	—44,244	13,547	57,891	71,438
Shall the city enter into a contract for the private construction and operation of an incinerator, subject to the right of purchase by the city	—36,338	27,740	64,078	91,818

The total vote for supervisors at the same election was 103,467.[9]

The garbage of San Francisco is now collected by scavengers at the expense of householders, and is burned in an old and much overworked incinerator owned by the city but operated by the scavengers. Some months ago, in a nuisance injunction suit, the court declared that the plant must be shut down, but withheld an order to that effect pending the outcome of proposals for some other means of disposal.

At the election on Nov. 6 two ordinances appropriating money for park purposes were defeated and one for establishing a municipal court system was approved.

Baron Furuichi Made Honorary Member of Am.Soc.C.E.

Baron Koi Furuichi, whose election as an honorary member of the American Society of Civil Engineers was briefly noted last week, is one of the leading civil engineers of Japan. He is president of the World Engineering Congress recently concluded in Tokyo and also president of the sectional meeting of the World Power Conference held at the same time. Born at Himeji, Japan, in 1854, he studied engineering in France from 1875 to 1880. His professional career includes service as director of the Japanese Public Works Bureau, Home Office; Vice-Minister of Communications; director-general of the Government Railways; director of the Seoul-Fusan Railway; and president of the Tokyo Subway Company. He was created a baron in 1919, and since 1924 has been a member of the Privy Council of the Japanese government. Baron Furuichi is at present an emeritus professor of the Tokyo Imperial University, having previously been dean of the engineering colleges of the university. He is also a member of the Imperial Academy.

The Business Outlook

The week's records show that basic industrial activity and general trade are so far holding up well in face of continued stock market liquidation. Steel ingot production averaged about 78 per cent of capacity, as against nearly 80 per cent in the preceding week and 86 per cent a year ago this time. This reduction is considerably more moderate than took place at this time last year, and operations are expected to hold fairly close to the present level for some weeks.

We expect the financial and business readjustment for the next four months to be slow, gradual and orderly. There will be a lot of price cutting, and automobiles are likely to be cold this winter. But the recession will be well cushioned by easier money and by well-controlled inventories in most lines, except raw materials. Early in 1930 business is likely to resume its normal stride.

—The Business Week, Nov. 9.

Five Killed by Explosion in Boston Water Tunnel

A shift boss and four of his crew were literally blown to pieces early Nov. 8 and two men were seriously hurt by a dynamite explosion at the heading 5,000 ft. west of shaft 7 of the 13-mile Wachusetts-Coldbrook tunnel being driven to augment Boston's water supply. The exact cause of the accident will probably never be known. According to one of the two survivors of the crew of seven, the entire face of the heading had been drilled and the center cut loaded and shot. After a wait of 25 minutes the crew started for the heading again, carrying about 100 lb. of cartridges primed with electric caps, enough to shoot the remainder of the heading. The foreman was ahead, carrying a lantern to light the way, as the lights had been removed for the cut shot. The two survivors of the blast had stopped to search for a couple of shovels.

The explosion occurred just as the foreman reached the heading. The two men bringing the shovels were knocked unconscious for a few minutes, and upon reviving crawled and stumbled toward the shaft. They were met halfway by men working at the surface, who, becoming alarmed by the intensity of the blast and by the fact that sufficient time had not elapsed after the firing of the cut shot to load and fire the remainder of the heading, had rushed down the shaft to investigate. Both of the survivors had their eardrums shattered and were badly bruised.

The accident brings the total fatalities of the work to twelve, seven men being killed in six previous premature explosions. Carleton & Rankin are subcontractors on the section where the explosion occurred. The West Construction Company is general contractor on the tunnel.

Secretary of Interior Disapproves of Rio Grande Project

The compact signed by the states of Colorado, New Mexico and Texas apportioning the waters of the Rio Grande among those states has been disapproved by the Secretary of the Interior in a letter sent to the chairman of the Senate committee on irrigation and reclamation recommending that the bill before the Senate approving that compact should not be passed. The Secretary states that his recommendation is based upon his opinion that the compact does not definitely settle the question of apportionment of the waters of the Rio Grande among the states interested, but to the contrary, provides for federal expenditures for the reimbursement of which no provision is made and of such a character as would create an undesirable precedent. The interstate river commissioner for New Mexico, Francis C. Wilson, states that the Secretary's objections are based on only the second part of the bill and that the first part, which contains the provision for the approval of the compact, is not within the province of the Secretary of the Interior.

LONG PILES SHIPPED ACROSS COUNTRY

Part of a shipment of 120-ft. Douglas fir piling being shipped from the Pacific Northwest for use in foundation work for the new Ford Motor Company plant at Edgewater, N. J. The order totals about 500 triple-carloads, and includes 7,500 piles 120 ft. long and 1,550 of lesser lengths. The average butt thickness of the piling is 22 in.; tops average 7 in.

Water and Sewage Conferences Held in Ohio

Two meetings of interest to sanitary engineers were held in the State of Ohio during the latter part of October. The third annual Ohio Conference on Sewage Treatment was held at Canton Oct. 22-23. The sewage-treatment works at Canton and at Salem were discussed in addition to topics of more general interest to engineers engaged in the problem of sewage treatment. On Oct. 24-25 in Columbus the ninth annual Ohio Conference on Water Purification paid special attention to the problem of the elimination of tastes and odors, and the sanitation of swimming pools.

Col. J. Franklin Bell Retires From Army

Approval has been given by President Hoover to the application of Lieut.-Col. J. Franklin Bell, engineer officer of the Ninth Corps Area at San Francisco, Calif., for retirement under the 30-year service law. He is to be relieved of duty Nov. 15 and is to be retired soon afterward. It is believed that Colonel Bell will accept an offer recently made him to become city manager of Covington, Ky.

Born in Pennsylvania in 1870, Colonel Bell was graduated from Washington and Jefferson College in 1898, from West Point in 1902 and from the Army Engineer School in 1910. He has been engaged in river and harbor projects in many parts of the country, having filled assignments in Albany, Jacksonville, New Orleans, Pittsburgh and the Philippines, and also as engineer commissioner of the District of Columbia. During the World War he held the rank of colonel in the national army, commanding the 24th Engineers in France from December, 1917, to March, 1918, and later being in charge of the district at Angers and the engineer depots at Gievres and Montierchaume. Later he was attached to the army of occupation, serving as executive officer for the chief of engineers of the 2nd army, as chairman of the board of disposition of German war materials and as assistant purchasing officer at Paris.

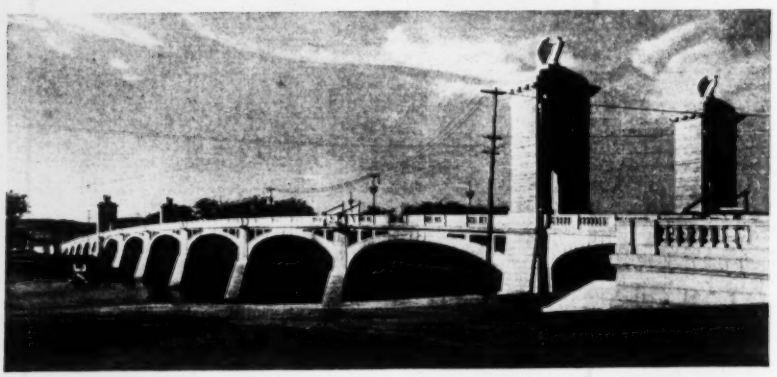
San Antonio to Develop City Plan

San Antonio, Tex., has appropriated $45,000 to be expended over a three-year period in the development of a comprehensive city plan, which will include an analysis of the growth of the city, plans for orderly expansion of the city as a whole and of the central business district, transportation facilities, recreation areas and city planning laws necessary to put the program into effect. The work will be under the direction of Harlan Bartholomew, consulting engineer, of St. Louis, Mo. His chief associate will be Earl O. Mills.

Colorado Utilities Commission Forbidden to Impose Penalties

That the state public utilities commission is not a court and has no right to impose fines has been held by District Judge Frank McDonough, Sr., in a decision rendered Oct. 29 in Denver, Colo. The contested case involved a $200 fine which had been imposed upon a bus operator for ignoring an order to stop operating between Denver and Golden, Colo.

MARKET STREET BRIDGE ACROSS SUSQUEHANNA RIVER BETWEEN WILKES-BARRE AND KINGSTON, PA.



The new 1,700-ft. multiple-arch concrete bridge spanning the Susquehanna River between Wilkes-Barre and Kingston, Pa., was formally dedicated and opened to traffic Sept. 25. The structure, built at a cost of $2,500,000 by the commissioners of Luzerne County, consists of twelve arches varying from 50 to 120 ft. between springing lines. Two trolley tracks are carried in the center of a 54-ft. roadway, which is flanked on both sides by a 17-ft. sidewalk. The bridge replaced an old structure and was constructed a half-width at a time to maintain traffic at all times. The late B. H. Davis, of New York, was consulting engineer on the bridge until his death in August, 1927, when he was succeeded by David A. Keefe, Athens, Pa. Carrere & Hastings were the architects, and Walter S. Rae, Pittsburgh, was the contractor.

City of Newark Rejects Low Bid on Pipe-Line Construction

After a delay of a week, the city commissioners of Newark, N. J., on Nov. 6 awarded the contract for constructing a water line from Eighth Ave. to the Elizabeth city line to the T. A. Gillespie Company, of New York, for $1,022,320, rejecting the low bid of $888,480 submitted by the International Excavating Company as illegal. According to the commissioners, the International company failed to include with its bid the name of the pipe manufacturer who was to furnish the material, and also failed to state the name of the engineer who would be in charge of the work, both of which were specifically asked for in the official call for bids.

The mayor and commission branded the International bid as unfair to the rest of the bidders, who had fully complied with bidding requirements, in that it would have permitted the shopping around for pipe prices after the bids were opened. The mayor and the entire commission were unanimous in voting for the rejection of the low bid, and claimed the International company could not do the work at the prices it submitted without the help of costly extras.

Missouri Valley Water-Works Men Meet at Sioux Falls

Factors of safety in water-works operation and hard water problems were the topics receiving the greatest amount of discussion at the meeting of the Missouri Valley section, American Water Works Association, held Nov. 6-8 at Sioux Falls, S. D. The former topic was provoked by a description by Earle L. Waterman, professor of sanitary engineering, University of Iowa, of a 35-hour shut-down of the plant at Albany, N. Y., due to a series of accidental break-downs last summer; also Jack J. Hinman, Jr., president of the parent society, in his remarks on tuning up the recently completed 8,000,000-gal.-daily filter plant for Barranquilla, Colombia, in which his first duty was to list all the items about the plant, from valves to chemicals, that should be kept on hand as spares. C. H. Koyl, 75-year-old veteran engineer of water service, Chicago, Milwakee & St. Paul Railroad, led the discussion on hard water treatment. He told in simple terms how the railroad water chemists and engineers had progressed to the point where there need be no scale or leaking of boilers and where the causes and condition of pitting are known and three workable methods of reducing and frequently preventing it are available. In 1928 there were 900 lime-soda plants treating 120,000,000 gal. daily, plus an equal amount treated by the soda-ash or zeolite method. From the 9,000 water stations of the railroads of the United States 900,000,000 gal. daily are furnished.

The round-table discussion revolved around rates and the different methods

for charging for fire service. The local iron-removal plant and novel collection wells with floating raft for the pumps installed in 1922 by Dabney H. Maury were described by F. G. Gordon, consulting engineer, Chicago, successor to Mr. Maury's practice. (See *Engineering News-Record*, Nov. 29, 1923, p. 882.)

Following a paper by Waldo W. Towne, director, division of sanitary engineering, South Dakota State Board of Health, on "Characteristics of South Dakota Water Supplies" the question came up of why a sharply defined area in the north central part of the state produced hot water artesian wells if the westward sloping Dakota sandstone strata collect these waters in the Black Hills region and deliver them eastward. E. P. Rothrock, state geologist, was sent for and stated that only one tenable theory had been advanced—namely, that the Dakota sandstone strata had been faulted in this comparatively local area and the trapped heat generated by the slips was transmitted to the artesian well water. R. E. Bragstad, city engineer, Sioux Falls, described the newly completed sewage-disposal plant wherein the sludge is separately digested and heated by burning the rich (800 B.t.u.) collected gas under a hot water boiler and circulating the water in the digestion tanks to maintain a temperature of 90 to 100 deg. F. E. B. Black, consulting engineer, Kansas City, described the filter plant now under construction at Tulsa, Okla., to treat the Spavinaw supply. One novel feature is the aeration in a flume by pumping air through filtros plates, thus saving head in the water.

Newly elected officers of the section are as follows: chairman, Thomas D. Samuel, Jr., Kansas City, Mo.; vice-chairman, H. V. Pedersen, Marshalltown, Iowa; director for parent organization, T. J. Skinker, engineer of distribution, St. Louis, Mo. Each state has representation on the board of the section: J. Chris Jensen, Council Bluffs, Iowa; H. L. Brown, Salina, Kan.; W. Scott Johnson, Jefferson City, Mo.; D. L. Erickson, Lincoln, Neb.; J. S. Nelson, Sioux Falls, S. D.

South Carolina Society of Engineers Organized

The South Carolina Society of Engineers was organized in Columbia on Nov. 7 by a group of the leading professional engineers of the state. Most of the large engineering organizations, four educational institutions and all sections of the state were represented at this meeting.

Officers were elected as follows: president, Harwood Beebe, consulting engineer, of Spartanburg; vice-president, John McCrady, consulting engineer, of Charleston; secretary-treasurer, L. A. Emerson, consulting engineer, of Columbia. Members of the board of directors in addition to these officers are: northwestern district, Dean S. B. Earle, of Clemson College; central district, T. Keith Legare, secretary of the state Board of Engineering Examiners, of Columbia; southeastern district, J. E. Gibson, manager and engineer, water department, Charleston.

The annual meeting of the society will be held some time in January at a place to be selected later.

WASHINGTON NOTES

By Paul Wooton
Washington Correspondent

New Orleans Bridge Case to Be Reconsidered—Rail-Water Joint Rates Call for Study

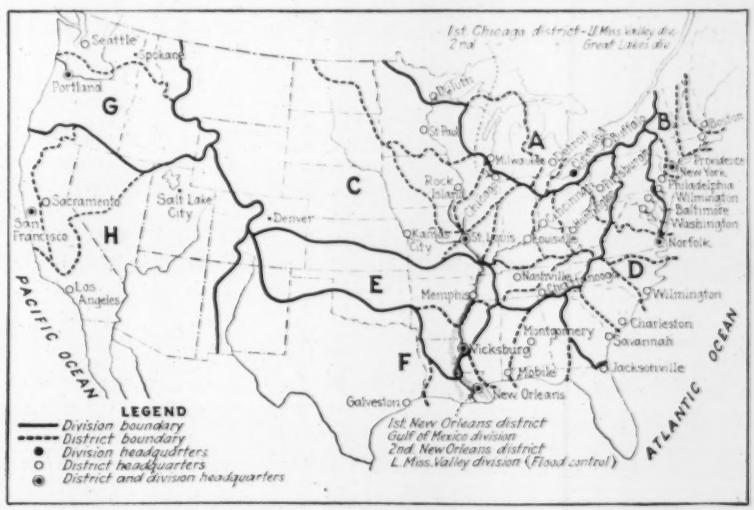
ALTHOUGH assurances have been given the backers of the Hero-Hackett bridge project at New Orleans that the War Department is still open-minded regarding the granting of an application, it is not believed that any new data sufficient to cause the army engineers to give up their previous objections have been submitted. One of the principal points to be studied in the reopened case is the matter of foundations for the heavy towers on each bank of the river. In previous investigations the army engineers were not satisfied that such heavy structures could be carried on soil such as exists on the immediate banks of the river. While highly responsible engineers in private practice stated that they had no doubt as to their ability to provide the necessary foundation, no technical information was submitted to show how they would do it. Engineers for the New Orleans bridge were thoroughly successful in providing adequate foundations for the piers of the new bridge at Charleston, S. C., where it is said foundation conditions are worse than those disclosed by borings at New Orleans.

Illinois Waterway

A meeting of the Rivers and Harbors Board of the Corps of Engineers has been called for the purpose of drawing up schedules for the completion of the government's portion of the Illinois waterway. Army engineers are surveying the state's portion of the waterway, which comprises a section from Lockport to Utica, in order to obtain information requested by the commerce committee of the Senate for the purpose of determining the advisability of authorizing the War Department to take over and complete the state's portion.

In discussing the matter recently, the Secretary of War pointed to the necessity of supplying Congress with facts and recommendations relative to the work on the waterway that has been undertaken by the State of Illinois, so that provision can be made in the pending river and harbor bill for the completion of the work should it be decided to nationalize the entire project. The state has exhausted the $20,000,000 bond issue which it raised for the work and will have difficulty in managing further financing, he said. Whether or not the nationalization of the whole waterway would place the government

REALIGNMENT OF U. S. ENGINEERS



Divisions and districts of the Corps of Engineers, U. S. Army, in accordance with the recent redistribution of territory are shown on this map. The official notation of divisions is as follows: (*A*) Great Lakes; (*B*) North Atlantic (includes Porto Rico); (*C*) Upper Mississippi Valley; (*D*) South Atlantic; (*E*) Lower Mississippi Valley; (*F*) Gulf of Mexico; (*G*) North Pacific (including Juneau, Alaska, district); (*H*) South Pacific (including Honolulu district). Three districts have headquarters in New York City, all in the North Atlantic division. There are two Chicago divisions, one attached to the Upper Mississippi Valley and the other to the Great Lakes division. In New Orleans there are also two district offices, attached to the Gulf of Mexico and Lower Mississippi Valley divisions, respectively. Cross-hatching in the Mississippi Valley indicates the jurisdiction of the Mississippi River Commission, pertaining to flood control.

engineers in direct charge of the diversion of water from Lake Michigan, he declined to say.

Railway Branch Recommended

Construction by the Pittsburgh & West Virginia Railway of a 6-mile branch tapping the Donora industrial district on the Monongahela River is recommended to the Interstate Commerce Commission in a proposed report on the carrier's application by C. V. Burnside, assistant director of the commission's bureau of finance. The line would feed into the Connellsville extension of the P. & W. V. at Pigeon Creek and connect at the other end with the Donora Southern near Donora, where the American Steel & Wire Company is located. Construction of the Connellsville extension connecting the Pittsburgh & West Virginia with the Western Maryland was approved by the commission on the ground that, in combination with the Wheeling & Lake Erie and possibly the Wabash, it would furnish a new and independent through route through the steel-producing territory between Baltimore and the Middle West. The extension to Donora is opposed by the Pennsylvania Railroad, but Assistant Director Burnside reports that denial of the Pittsburgh & West Virginia's application would deprive the Connellsville extension of one of the principal sources of traffic it was designed to serve. The Donora branch would be a single-track heavy-rail line costing approximately $1,400,000.

Barge Line Rates

Establishment of specific joint rail-barge-rail rates via the federal barge lines and connecting railroads is sought in a petition filed by the Inland Waterways Corporation with the Interstate Commerce Commission. Application of a uniform differential of 20 per cent also is urged. In explanation, the petition states that the Inland Waterways Corporation has found it impossible to comply with the formula prescribed in the commission's order of April 8, 1929, for constructing rail-barge-rail rates, and after several months of negotiation with the railroads is convinced that rail-barge-rail rates can best be determined by dealing specifically with each of the territories involved. Employing 10 and 20 per cent differentials as required in the commission's formula, it is argued, would cause unwarranted and extensive discriminations. This variation was based on the theory that as the haul increases the economy effected in the water portion of the haul decreases.

As an initial step, the petition of the Inland Waterways Corporation requested that the commission establish joint rail-barge-rail rates between points in central freight association and western trunk line territories on the one hand, and points in Texas and Louisiana on the other, such rates to be constructed by deducting from the joint all-rail rate, between point of origin and destination, 20 per cent of the lowest all-rail rate, between the ports that the traffic is borne by the barge line.

Personal Notes

EDGAR JADWIN, lieutenant-general, U. S. Army, who retired recently as Chief of Engineers, has been retained as consulting engineer of the New Jersey Meadows Commission.

L. R. LOHR, formerly major in the Corps of Engineers, U. S. Army, and secretary of the Society of American Military Engineers, has been appointed manager and engineer of the Chicago World's Fair, in charge of preparations for the proposed exhibition in 1933.

MAURICE J. LONERGAN, principal assistant engineer, Iowa State Board of Health, has been appointed sanitary engineer of the Colorado State Board of Health, succeeding DANA E. KEPNER. For two years after graduation from University of Iowa in 1923 he was assistant superintendent of the Mason City (Iowa) water-works.

WALDO W. TOWNE, until recently with the Iowa State Board of Health, has been appointed director of the division of sanitary engineering, South Dakota State Board of Health.

FRANK W. HERRING, formerly an assistant engineer in the Department of Water Supply, Detroit, Mich., has joined the editorial staff of *Engineering News-Record*. Mr. Herring is a graduate of Johns Hopkins University, class of 1920.

RAYMOND B. TRAVER, superintendent of highways for Onondaga County, N. Y., does not intend to resign from that position, as was erroneously stated in our issue of Oct. 31, p. 710.

H. L. RIPLEY, corporate and valuation engineer of the New York, New Haven & Hartford Railroad, at Boston, Mass., has been appointed assistant to the vice-president of that road, with offices at New Haven, Conn.,

DANIEL E. DAVIS has been appointed division engineer of the newly created division 12 of the Ohio highway department, comprising Cuyahoga County, with headquarters in Cleveland. Mr. Davis was resident engineer of Athens County from 1923 to 1925 and was then appointed state highway inspector in Cuyahoga County.

A. H. HEITZLER, for the past five years division manager at Lorain, Ohio, for the Ohio Public Service Company, has been made superintendent of the electric department of the Public Service Company of Colorado, with headquarters in Denver. He succeeds HARRY H. KERR, promoted to the superintendency of electric operation for the Toledo Edison Company, with headquarters at Toledo, Ohio.

ANDREW T. LONG, rear admiral, U. S. Navy, has been nominated by President Hoover, on recommendation of the State, Navy and Commerce departments, as the candidate of the U. S. government to fill the vacancy on the board of directors of the International Hydrographic Bureau at Monaco which resulted from the death of Rear Admiral A. P. Niblack. The election will take place about Feb. 1.

Obituary

WILLIAM OTTENBURG, pioneer railroad contractor, who more than 60 years ago secured a contract on the construction of the Delaware & Hudson Railroad from Whitehall to Plattsburg, N. Y., died in Clemons, N. Y., on Nov. 6, in his 80th year.

HERBERT F. PIERCE, of Boston, Mass., a member of the civil engineering firm of Pierce & Barnes, died on Oct. 27. During the World War Mr. Pierce had charge of the government housing program at Bath, Me. He was 63 years old.

BERNARD D. BARKER, chief engineer of the Cook County department of highways, Chicago, since 1912 and a member of the engineering firm of Barker, Flavin, Shields & Wallace, Chicago, was killed in an automobile grade-crossing accident on Nov. 7. Mr. Barker was an associate member of the American Society of Civil Engineers.

CARL F. ELLERS, head of a group of engineers making a survey near Ocean City, Md., for the U. S. Coast and Geodetic Survey, was found dead on Nov. 9 in a government truck in the woods with a bullet through his temple and a revolver in his hand. The states attorney, after an investigation, pronounced it suicide. Mr. Ellers' home was in Iowa. He was 30 years old.

WILLIAM BUTLER JONES, formerly superintendent of public works in Woburn, Mass., died on Nov. 4, aged 64 years. Mr. Jones was a graduate of West Point. He was city engineer of Woburn from 1889 until 1920, when he was appointed superintendent of public works. In 1927 he retired because of poor health, and had since been a consulting engineer.

Society Calendar

AMERICAN ROAD BUILDERS ASSOCIATION, Washington; annual convention, Atlantic City, N. J., Jan. 11-18.

AMERICAN WOOD-PRESERVERS ASSOCIATION, Chicago; annual meeting, Seattle, Wash., Jan. 28-30.

INTERNATIONAL CITY MANAGERS ASSOCIATION, Chicago; annual convention, Forth Worth, Tex., Nov. 20-23.

NATIONAL CRUSHED STONE ASSOCIATION, Washington; annual convention, Cincinnati, Ohio, Jan. 20-23.

NATIONAL PAVING BRICK MANUFACTURERS ASSOCIATION, Washington; annual meeting, Chicago, Ill., Dec. 4-6.

NATIONAL SAND AND GRAVEL ASSOCIATION, Washington; annual meeting, Memphis, Tenn., Jan. 28-30.

AT A MEETING of the board of directors of the Pacific Northwest Section of the American Water Works Association in Portland, Ore., it was decided to hold the next annual meeting in that city April 24-26, 1930. Announcement was made at the meeting that the section was the winner of the Hill cup, awarded each year to the section showing the largest percentage in gain in membership.

Construction Equipment and Materials

Highway Manufacturers to Expand Activities

As the first step in the development of additional activities in behalf of its members, the manufacturers division of the American Road Builders Association, through its executive committee, has appointed William Ogden to the post of manager. Mr. Ogden will devote his full time to the work of the division, with headquarters in Washington, D. C. He will develop and carry on, under the supervision of the executive committee, activities relating to the problems confronting manufacturers in this field.

Concrete Pavement Yardage

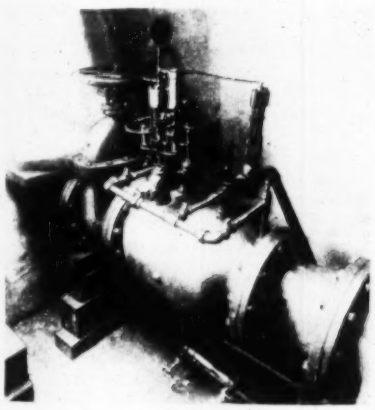
There is given below a tabulation of concrete pavement yardage awarded during the month of October, 1929, and also the total number of square yards awarded for roads, streets and alleys in the United States for the year 1929 to date as of Nov. 2:

	Sq.Yd. Awarded During October, 1929	Sq.Yd. Awarded From Jan. 1, 1929, to Nov. 2, 1929
Roads	9,157,203	84,302,889
Streets	3,676,684	39,418,216
Alleys	317,518	3,122,789
Totals	13,151,405	126,843,894

New Developments

Hydraulic Valve Used in Pressure Regulator

For conversion of a variable upstream pressure into a constant and adjustable lower downstream pressure in water-supply systems the Larner-Johnson hydraulic valve, widely known in the water-power and irrigation field, has been adapted to pressure regulator service by the Pelton Water Wheel Company, of San Francisco, Calif. Because of its hydraulic balance the Larner-Johnson valve can be built in the largest commercial sizes.



PRESSURE REGULATOR IN SERVICE IN SAN FRANCISCO

The valve body is cylindrical in shape and forms a direct straight-flow portion of the pipe line in which it is placed. The operating cylinder is concentric with the body and is supported within it by integral struts, forming an annular passage for the flow of water. The operating cylinder, closed at the downstream end, contains a differential area plunger which seats against a ring in the upstream end of the valve body when in the closed position. Movement of this plunger results from hydraulic forces on the cone and within the operating chambers of the cylinder. All principal elements of the valve are circular, resulting in minimum distortion.

Operating pressure for the cylinder is taken from the upstream conduit through external duplex strainers and a throttling orifice. Discharge from the operating chamber passes into the downstream conduit, being regulated by a control pilot, the position of which is fixed by the downstream pressure. This pilot is adjustable and operates over a small range in pressure corresponding to the changes in demand. An auxiliary pilot, actuated by the same downstream pressure, produces emergency closure should the downstream pressure reach an adjustable amount slightly above that of the control pilot setting.

To prevent any tendency for periodic surges, a compensator is provided with dual dashpots and adjustable bypass. This mechanism prevents sudden plunger movements, but permits the normal relations to be promptly performed at an adjustable rate in conformance with the surge period of the system. A relief valve protects the compensator against undue pressure which might arise with abnormal operation or adjustments.

Drag Scraper Controlled by Hydraulic Cylinder

Accuracy in grading and spreading is the outstanding advantage of the Ateco hydraulic dirt mover manufactured by the American Tractor Equipment Company, of Oakland, Calif. This is designed for use with any make of tractor, and is made in two sizes, the S-8, with a rated capacity of 2 cu.yd. and an 8-ft. bowl, and the S-10, which has a bowl 10 ft. wide and carries 4 cu.yd. of material. The two main portions of the drag scraper, the bowl and the wheel assembly, are joined by a pin connection, the angle between the two parts being regulated by means of a hydraulic cylinder actuated by an oil pump operated from the power take-off of the tractor. Thus the tractor operator can raise and lower the wheels and

POSITION OF SCRAPER BOWL SET BY HYDRAULIC CYLINDER

set the bowl at any desired height. As the wheels travel on the finished grade, the cutting depth can be set with unusual accuracy. When loading, the wheel assembly can be raised clear of the ground, its weight aiding in cutting hard material. When necessary, additional weight can be obtained by filling the hollow wheels with water. Hyatt roller bearings are used to minimize friction, being fully protected from dirt and water by dust caps and felt washers.

Truck-Mounted Pump Cleans Sewer Catch Basins

Removing trash from sewer catch basins is the function of a machine being marketed by the Four Wheel Drive Auto Company, Clintonville, Wis., and known as the FWD Auto Eductor. The pumping unit used is made by the Elgin Sweeper Company, Elgin, Ill. A watertight body with a capacity of 1,200 gal. is mounted on a six-wheel truck in combination with a pump driven by a separate motor. In the cleaning operation, two telescopic pipes are lowered into the catch basin, the pump is started and water is forced down one of the pipes and through a nozzle which reduces the 3-in. stream to 1⅛ in. This nozzle points upward into the second pipe and creates a suction which draws the trash from the catch basin into the moving stream, which carries it upward into the body of the truck. The dirt and trash settle to the bottom of the tank, while the water is worked through baffle plates to the front end of the body and re-used.

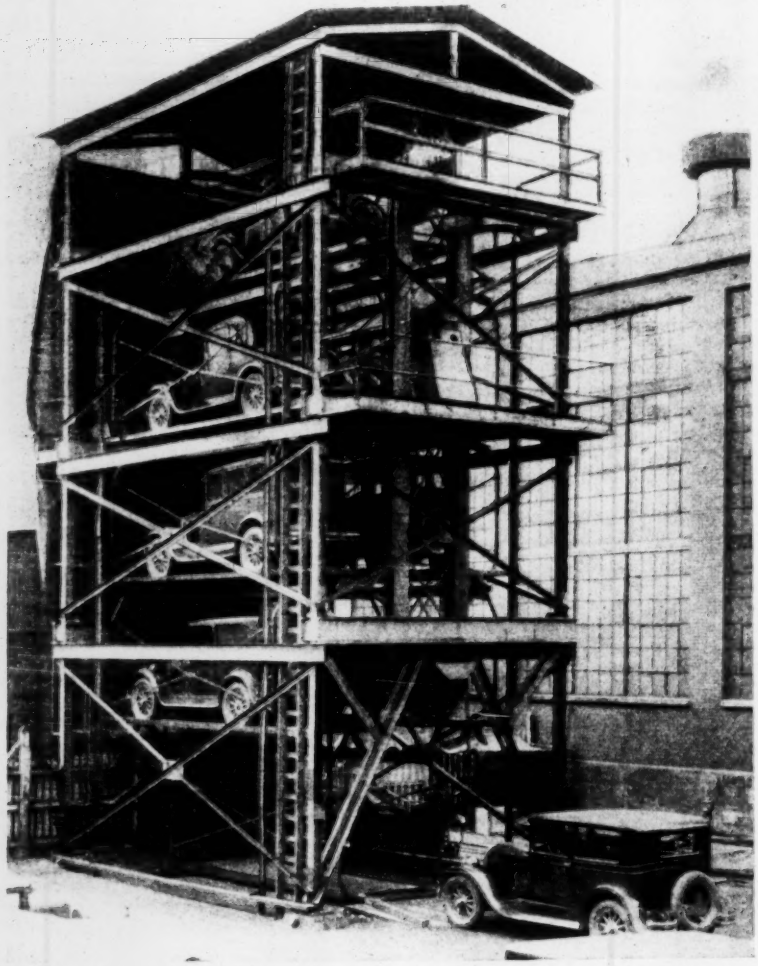
EDUCTOR TRUCK

Portable Welding Outfit Uses Six-Cylinder Motor

Power for an electric arc-welding machine announced by the General Electric Company, Schenectady, N. Y., is derived from a six-cylinder gasoline engine developing 39 hp. at 1,440 revolutions per minute. This motor is totally inclosed in a sheet-metal housing. It is equipped with high-tension magneto, air cleaner and a gasoline tank of 10-gal. capacity. The welding generator is a ball-bearing, self-excited, single-operator machine rated at 300 amp., 1 hour, 50 deg. C., with a current range of 90 to 375 amp. A current-reducing resistor is provided, by means of which welding currents down to 25 amp. may be obtained. A self-adjusted stabilizing reactor automatically steadies the arc. The complete outfit is mounted on a welded structural steel base and may be made portable by the addition of wheels or skids.

MOTOR-DRIVEN WELDING OUTFIT

MACHINE FOR PARKING AUTOMOBILES DEVELOPED



Storage of motor cars in small open spaces is made possible by a new machine developed by the Westinghouse Electric & Manufacturing Company, East Pittsburgh, Pa., which consists of two endless chains passing over large wheels at the top and bottom of the installation. Platforms are suspended between these chains, each of which provides space for one automobile. No floors are necessary; the cars remain on the machine until called for. The device can readily be built into either old or new buildings, to any height desirable. While a certain amount of unbalanced loading is of course inevitable, it is planned to minimize this by loading cars into alternate spaces until 50 per cent of the capacity has been utilized.

Business Notes

AMERICAN LOCOMOTIVE COMPANY, New York City, has acquired the McIntosh & Seymour Corporation, of Auburn, N. Y., manufacturer of diesel engines, which will continue under its present management as a subsidiary of the American Locomotive Company.

MCKIERNAN-TERRY CORPORATION is the new corporate name of the McKiernan-Terry Drill Company, New York City, which has acquired ownership of the National Hoisting Engine Company, Harrison, N. J., and Steele & Condict, Inc., of Jersey City, N. J. The latter two will preserve their identity and will be operated as divisions of the McKiernan-Terry Corporation. Executive offices will be concentrated in New York City.

NATIONAL LUMBER MANUFACTURERS ASSOCIATION, Washington, D. C., has placed J. R. Rapp, formerly with its New York field office, in charge of the Pittsburgh office, succeeding Harold S. Crosby, who has been transferred to Kansas City.

AMERICAN ASPHALT PAINT COMPANY, Chicago, Ill., has purchased a factory at Kankakee, Ill., with approximately 100,000 sq.ft. of floor space, which will be used in the manufacture of asphaltic products.

FATE-ROOT-HEATH COMPANY, Plymouth Locomotive Works, Plymouth, Ohio, has completed construction of a new factory unit which will double its present output of gear-driven gasoline and diesel locomotives.

STANTON IRONWORKS COMPANY, LTD., near Nottingham, England, has decided to build an addition to its English plant for the manufacture of de Lavaud spun iron pipe.

JOSEPH T. RYERSON & SON, INC., has purchased the business, equipment and stock of the Penn-Jersey Steel Company, Camden, N. J.

INDEPENDENT PNEUMATIC TOOL COMPANY, Chicago, Ill., has appointed Joseph F. Fletcher manager of its Philadelphia office to succeed the late A. L. Schuhl. Mr. Fletcher, who was formerly manager of the Buffalo office, will be succeeded there by W. O. Becker.

ALEXANDER L. SCHUHL, manager of the Philadelphia office of the Independent Pneumatic Tool Company since 1924, died Oct. 20 of pneumonia.

HENRY AYLETT COLES, Atlanta district manager of the Westinghouse Electric & Manufacturing Company, died recently in New Orleans, La., of heart failure. Mr. Coles had been Westinghouse representative in Atlanta since 1897.

Costs and Contracts

E. N.-R. Index Numbers

Cost			Volume		
Nov. 1, 1929		**208.46**	**Oct., 1929**		**273**
Oct. 1, 1929		206.32	September, 1929		340
Nov. 1, 1928		209.46	October, 1928		363
Average, 1928		206.78	Average, 1928		287
Average, 1927		206.24	Average, 1927		263
1913.....		100.00	1913.....		100

This Week's Contracts

Heavy construction contracts, reported by *Engineering News-Record* in the week of Nov. 14, with some comparisons, total as follows:

(In Thousands of Dollars)

	Nov. 14, 1929	Nov. 7, 1929	Nov. 15 1928
Buildings:			
Industrial.....	$9,270	$9,845	$5,905
Commercial....	27,277	46,435	31,334
Streets and roads.	12,087	6,052	10,872
Other eng. constr.	17,750	14,887	60,833
Total.....	$66,384	$77,217	$108,944
Total, all classes, Jan. 1 to Nov. 14:			
1929.....			$3,999,140
1928.....			3,268,409